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# Urban Stormwater Management and Technology: Case Histories

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# Urban Stormwater Management and Technology

## Case Histories

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This report is the third in a series on urban stormwater and combined sewer overflow management. It presents 12 case histories representing most promising approaches to stormwater control. The case histories were developed by evaluating completed and operational facilities or ongoing demonstration projects that have significant information value for future guidance. Essential elements of the case history evaluations cover (1) approach methodology, (2) design considerations, (3) costs, (4) effectiveness, and (5) environmental and socioeconomic impacts. Eight of the case histories assess Best Management Practices (BMPs) and expand the data base on source control methodology, focusing principally on planning and storage alternatives. Special considerations are given to flood and erosion control measures also having a dual benefit of stormwater control. The project sites evaluated are Bellevue, Washington; Montgomery County, Maryland; Lake Tahoe, California; The Woodlands, Texas; Orange County, Florida; San Jose, California; Middlesex County, Connecticut; and Boulder, Colorado. The remaining four case histories evaluate the control of combined sewage overflows and document a systems approach in applying unit process alternatives. The effectiveness and unit costs of storage and treatment processes are presented, together with evaluations of areawide and systemwide integration of these technologies. Storage, the key element of an integrated approach, can involve storage/wet-weather treatment or storage/dry-weather treatment, or both. The project sites are Seattle, Washington, Saginaw, Michigan; Mount Clemens, Michigan; and Lancaster, Pennsylvania.

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# URBAN STORMWATER MANAGEMENT AND TECHNOLOGY: CASE HISTORIES

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# FOREWORD

The Environmental Protection Agency was created because of increasing public and government concern about the dangers of pollution to the health and welfare of the American people. Noxious air, foul water, and spoiled land are tragic testimony to the deterioration of our natural environment. The complexity of that environment and the interplay between its components require a concentrated and integrated attack on the problem.

Research and development is that necessary first step in problem solution and it involves defining the problem, measuring its impact, and searching for solutions. The Municipal Environmental Research Laboratory develops new and improved technology and systems for the prevention, treatment, and management of wastewater and solid and hazardous waste pollutant discharges from municipal and community sources, for the preservation and treatment of public drinking water supplies, and to minimize the adverse economic, social, health, and aesthetic effects of pollution. This publication is one of the products of that research; a most vital communications link between the researcher and the user community.

As the nation moves closer to its goals of clean water by implementing programs that control urban stormwater runoff and combined sewer overflows, the experience gained from operating full-scale control facilities becomes invaluable. Using this experience in future planning and design is an essential step in implementing cost-effective systems for urban stormwater pollution control.

Francis T. Mayo
Director
Municipal Environmental Research Laboratory

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# SECTION 1

# INTRODUCTION

Over the past decade, the technology available for controlling urban runoff and combined sewer overflows has progressed from state-of-the-art demonstration projects, designed to test the feasibility of individual treatment processes, to prototype facilities using an areawide systems approach. The systems approach integrates several technologies to achieve effective control over large service areas. However, even with the detailed planning studies, problem characterization studies, and process evaluations conducted for these facilities, information on the receiving water quality benefits and cost effectiveness of the control facilities has been limited and is nonexistent on a nationwide basis for the potential user community.

Many of our larger cities with combined sewer overflow problems have embarked on massive receiving water restoration and cleanup projects, as a result of the Federal Water Pollution Control Act Amendments of 1972 (PL 92-500) and the Clean Water Act Amendments of 1977 (PL 95-217). Many communities are also constructing small-scale urban runoff controls, often combining multiuse purposes of flood and erosion/sediment control to reduce the impacts from urban runoff on receiving waters. Although no technology-based treatment requirements exist for storm and combined sewer overflows, federal funding for the study, design, and construction of control or treatment facilities is contingent on evaluations showing them to be cost effective, within the scope of receiving water quality improvements or benefits received. In other words, the marginal costs of a proposed control system must be consistent with the expected marginal benefits. Costs should also include both socioeconomic and environmental costs and should be compared either quantitatively or qualitatively to pollutant reduction, water quality improvement, and improvement in beneficial uses.

Nationwide costs of over $28 billion (ENR 3000) to abate pollution from combined sewer overflows and achieve recreation objectives for receiving waters in the country's 1,600 communities with combined sewers were estimated in the EPA's 1978 Needs Survey [1]*. This estimate represents about a two-fold reduction from the 1974 needs estimate because the 1978 survey considered assimilation capacities of receiving waters and use of a mix of technologies reflecting cost-effective solution planning. Both estimates, however, were based on simplified assumptions of system performance, costs, and effectiveness, and are subject to continued refinement as actual operating data and information become available. To establish better estimates, it is imperative that the cost effectiveness of operating full-scale control/treatment facilities be evaluated.

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*References are listed by chapter at the end of the report.

This report, through a series of case histories, identifies the most promising control alternatives, reevaluates the performance, and assesses the costs and effectiveness of selected, constructed and operating urban runoff and combined sewer overflow control facilities. Additional goals of this study are identifying receiving water, socioeconomic, and environmental impacts and benefits.

## URBAN STORMWATER MANAGEMENT

The urban stormwater problem is characterized by surface runoff events that are unpredictable and highly time and location variable. Control strategies, therefore, need to be planned, evaluated, and designed subject to local stormwater characteristics. Guidance for alternative approaches, methodologies, and processes used on a nationwide basis provides a starting point and first-cut evaluation for developing local control programs.

### State-of-the-Art Studies

Nationwide evaluations of urban stormwater management approaches have been documented in a number of reports sponsored by the EPA Storm and Combined Sewer Section (Edison, New Jersey 08817) [2, 3, 4, 5]. Two of these reports document the advancement of the state-of-the-art of urban runoff and combined sewer overflow technology. The first report, URBAN STORMWATER MANAGEMENT AND TECHNOLOGY: An Assessment, completed in 1974, presented a compendium of information on control projects, unit processes, user assistance tools, evaluation procedures, and problem characterization [5].

The second report, URBAN STORMWATER MANAGEMENT AND TECHNOLOGY: Update and Users' Guide, completed in September 1977, presented an update and more detailed evaluation of the original report [4]. Data and information on many projects either under construction or just completed and not available for the first report were incorporated in the update to expand and normalize the data base, evaluate and screen approach methodologies and user assistance tools, and present more detailed and up-to-date evaluations of control system operation, performance, and costs.

### Needs

Several major conclusions and needs become apparent after reviewing the state-of-the-art advancement of the past 10 years:

- Although many demonstration and full-scale projects had been implemented, performance information was limited, particularly for control systems integrating several control strategies.

- The cost data base was insufficient for accurately estimating full-scale system costs or for evaluating cost effectiveness.

- Detailed information on the design, performance, and cost of source control strategies was lagging behind the information available on combined sewage overflow strategies.

- The link between receiving water quality/benefits and stormwater abatement programs was available in theory only and unsupported by monitoring data.

Establishing a credible link between receiving water quality/benefits and abatement programs is the key milestone to properly evaluate the feasibility and cost-effectiveness impacts, nationally and locally.

Trends in the application of stormwater technology solutions have been identified from reviews of the state-of-the-art literature and existing facilities. These solutions are generally considered essential and most promising elements in areawide/systems control approaches and are evident in both urban runoff source controls (low/nonstructural solutions) and combined sewage overflow controls (end-of-pipe/structural and source control solutions).

## CASE HISTORIES

As a continuing process of updating and disseminating information on urban stormwater state-of-the-art technology, a nationwide site search was conducted to identify those completed and operating projects representative of a most promising control approach. Most of the selected projects presented as case histories have been identified in the prior technology assessments [4, 5], and are reevaluated in greater detail here to take advantage of recent operating experience. Since many of the case histories dealing with source controls or Best Management Practices (BMPs) have not previously been reported, this subject is covered more fully in this report to advance the state-of-the-art knowledge.

### Site Search

A nationwide site search was initiated with 135 letters of inquiry seeking to identify potential projects for in-depth case study and evaluation. These letters were sent to local and regional EPA administrators, 201 and 208 project directors, state environmental officials, state water resource research institutions, county and city public works departments, and private consultants. Each respondent was asked to nominate candidate projects for in-depth evaluation under three general categories:

1. Completed and operating BMPs with potential multipurpose benefits.
2. New and ongoing BMP projects including demonstration projects and studies.
3. Completed and operating combined sewer overflow facilities.

A total of 41 responses to the letter of inquiry were received, of which 35 projects were identified as potential candidates for case study evaluation. These projects are summarized in Table 1.

## Table 1. SUMMARY OF POTENTIAL CANDIDATE BMP AND COMBINED SEWAGE OVERFLOW PROJECTS IDENTIFIED IN THE NATIONWIDE SITE SEARCH

| Project location | Type of control[a] | Project description |
|---|---|---|
| Gaithersburg, Md. | BMP | Northcreek Lake, Montgomery Village - 5 acre lake, controls peak flows from 2 and 10 year storms, 302 acre drainage area |
| Montgomery County, Md. | BMP | Spring Lake - 5 acre lake (25 acre-ft permanent pool), 130 acre drainage area |
| Baltimore, Md | BMP | Porous pavement grasscrete 2 acre parking lot |
| Occoquon Watershed, Va. | BMP | Urban runoff control demonstration of several non- and low-structural controls in a developing urban area, (source storage and infiltration), developed land use/runoff pollution relationships |
| Virginia Polytechnic Institute, Blacksburg, Va. | BMP | Runoff coefficient studies with various porous pavement materials (laboratory scale) |
| Prince William County, Va. | BMP | Lake Ridge residential development - five stormwater detention basins, 1,000 acres drainage area |
| Lynchburg, Va. | BMP | Infiltration/inflow study on portion of combined sewer system and retention pond |
| Orange County, Fla. | BMP | Several constructed and operating controls - percolation ponds, swales, detention/sedimentation, fabric filters, underdrains, and street sweeping |
| Concord, Mass. | BMP | Porous pavement parking lot at Walden Pond |
| Wallingford, Conn. | BMP | Debris/siltation basins, erosion and sedimentation control plans for subdivision and industrial park |
| Middlesex County, Conn. | BMP | Percolation basin, detention/sedimentation basin, and dry-well infiltration system serving industrial and residential developments |
| I-84, Conn. | BMP | Debris basins, siltation ponds, temporary and permanent seedings, and detention ponds |
| Marlboro, Conn. | BMP | Stream belt zoning regulations |
| Brooklyn, Conn. | BMP | Stream belt zoning regulations |
| Minneapolis, Minn. | BMP | Harriet Lake - vacuum street sweeping for nutrient removal; Lake of the Isles - first flush diversion of stormwater to sanitary sewer |
| Rochester, N.Y. | BMP | Study of source control practices - surface flow attenuation, porous pavement, erosion controls, chemical use restrictions, infiltration/inflow control, regulator improvement, and friction reduction and hydraulic improvement |
| Arlington Heights, Ill. | BMP | Twelve detention facilities planned or built, multi-use, largest is a 100 acre-ft reservoir serving 717 acre drainage area |
| Des Moines, Iowa | BMP | Land use control, detention storage, and floodwater control |
| The Woodlands, Tex. | BMP | Newly constructed town using porous pavement, natural drainage infiltration, detention storage, and land use planning |

## Table 1 (Concluded)

| Project location | Type of control[a] | Project description |
|---|---|---|
| Denver, Colo. | BMP | Source storage projects - rooftop storage, mall depression storage |
| Boulder, Colo. | BMP | Ordinances controlling runoff, onsite detention basins, rooftop storage, drainage construction or improvements |
| Davis County, Vt. | BMP | Countywide ongoing program - detention and siltation ponds, and instream drainageway storage; emphasis on multiuse; local financing program |
| Stateline, Nev. | BMP | Low structural erosion controls on unstable soil on owner-operated land parcels |
| South Lake Tahoe, Calif. | BMP | Heavenly Valley Ski Area drainage - slope regrading, revegetation, and drainage improvements |
| South Lake Tahoe, Calif. | BMP | Street drainage improvements and erosion control practices |
| Lake Tahoe Basin, Calif. | BMP | Unpublished study for erosion control methods and institutional modifications to reduce runoff |
| Lake Tahoe Basin, Calif. | BMP | Erosion control demonstration project Northstar Ski area and Rubicon properties - slope stabilization, revegetation, land use planning, and erosion controls; collected receiving water quality data |
| San Jose, Calif. | BMP | Demonstration project on street sweeping effectiveness and receiving water quality |
| Bellevue, Wash. | BMP | Low- and non-structural controls - source detention ponds, instream storage, and stormwater utility approach |
| King County, Wash. | BMP | Low- and non-structural controls with multiuse benefits and public educational program |
| Philadelphia, Penna. | CSO | Inflatable dams creating inline storage in combined sewer systems |
| Mattoon, Ill. | CSO | Diversion structures and conversion of sanitary treatment plant to provide primary sedimentation and disinfection of up to two times sanitary flow capacity |
| Kankakee, Ill. | CSO | Offline storage/sedimentation with screening and disinfection |
| Madison Heights, Mich. | CSO | Inline storage with 190 acre-ft of storage, disinfection, pumping, flushing facilities |
| New Boston, Ohio | CSO | Modified trickling filters to allow use of inflow as a substitute for filter recirculation, 1.7 Mgal/d flow |

[a] BMPs may also include source controls used on combined sewer systems to reduce flow volumes or pollutant loads; CSO controls are structural solutions to combined sewer overflow problems.

In addition, a number of combined sewage projects identified in prior studies were reviewed, screened, and contacted for potential case study evaluation [4]. These projects are summarized in the following:

- Saginaw, Michigan - Storage/sedimentation
- Seattle, Washington - Inline storage
- Mount Clemens, Michigan - Storage/transport, primary treatment, biological lagoons
- Lancaster, Pennsylvania - Swirl concentrator
- Oil City, Pennsylvania - Sedimentation/dual-use microscreening
- Milwaukee, Wisconsin - Storage/sedimentation
- Racine, Wisconsin - Screening/dissolved air flotation

## Project Monitoring

Because of the limited amount of operating data available for stormwater control facilities, funds were set aside to perform small sampling programs on selected facilities. The monitoring was designed to fill data gaps and develop a base of information for our case history evaluation.

A monitoring program was conducted in Saginaw, Michigan, at the Hancock Street storage/sedimentation facility to provide performance and operation data. Eleven storms were sampled between May and September 1978, in cooperation with the city. Influent combined sewage flow sedimentation performance and system pollutant load reduction were evaluated from the data. Sample analysis was performed for suspended solids, volatile suspended solids, $BOD_5$, COD, lead, zinc, chromium, total nitrogen, and total phosphorus. Evaluations of influent flow volumes, hydraulic loading rates, and effluent overflow volumes were also made.

## Report Format

Twelve stormwater and combined sewage overflow projects were selected for in-depth evaluation. Each case history is generally organized along the following outline of topics:

- Project description
- Performance
- Costs
- Impacts

The case history reports or sections are compiled under two major parts in this report. Part 1 contains eight case histories dealing with BMPs, and Part 2 contains four case histories describing combined sewage overflow control approaches.

### Part 1 - BMPs--

The BMP case histories cover subjects including: legislation/ regulation, land use planning, enforcement and funding approaches, and application of low and nonstructural source controls. Although many of the case histories are not limited to discussion of single subjects, several case histories describe demonstrations of single technologies or are representative of applications of source controls as unit processes rather than systemwide or areawide control approaches. The case histories highlight the most promising technology applications. The sequence and major topic of each case history section are as follows:

1. Bellevue, Washington - Approach methodology (public utility approach)
2. Montgomery Co., Maryland - Approach methodology (regional approach)
3. Tahoe, California - Planning, erosion controls
4. The Woodlands, Texas - Planning, urban runoff controls
5. Orange Co., Florida - Control description (detention)
6. San Jose, California - Control description (street sweeping)
7. Middlesex Co., Connecticut - Control description (onsite private controls)
8. Boulder, Colorado - Control description (flood/erosion controls)

### Part 2 - Combined Sewer Overflow Controls--

The sequence and general topic of the combined sewer overflow control case histories is as follows:

1. Seattle, Washington - Inline storage
2. Saginaw, Michigan - Storage/sedimentation
3. Mount Clemens, Michigan - Storage/sedimentation/treatment lagoons
4. Lancaster, Pennsylvania - Swirl concentrator

With the exception of the Lancaster case history, which describes a full-scale demonstration of a unit process, the case histories represent a description of full-scale systems approaches for areawide solutions.

### Summary and Recommendations--

The Summary and Recommendations sections of the report not only present the major findings of the studies, but also serve as a state-of-the-art continuation, update, and guidance document to present overall technology assessment for BMPs and combined sewer systems. The information represents a compendium of experience from operating full-scale systems that can serve as guidance for future planning processes, establishing priorities of control versus benefits, and cost-effective system selection.

## Metrics

Units of measurement in this report are metric units (SI) with U.S. customary units following in parenthesis. All values in the figures and tables are presented in U.S. customary units for consistency with the previous state-of-the-art reports.

A list of conversion factors is also presented at the end of the report for the reader's convenience in converting values between the U.S. customary and the SI.

## Cost Index

All costs contained in this report are based on or have been converted to an ENR 3000 base (1913 = 100). Where applicable, these costs have been regionally adjusted by city and do not simply represent an average nationwide base adjustment.

# SECTION 2

## SUMMARY

A large number of small communities and almost every major metropolitan area are implementing programs that use storage to control stormwater. Many controls have already been constructed and are currently in operation. Although storage is the most important element in a stormwater control system, other methods are also representative of most promising technology and have been applied individually to correct specific stormwater problems.

This section presents the results and conclusions of the 12 case history evaluations and highlights the approach methodology, design considerations, costs, and effectiveness of alternative control measures. The selected case histories represent a technology update of the most promising control approaches for both BMPs and combined sewer overflow controls for future guidance.

BMPs are gaining national prominence as a cost-effective solution to urban runoff problems and may also help to correct combined sewer overflow problems. However, information on the design, cost, and effectiveness of BMPs is limited in comparison with the information available on combined sewer overflow controls. Therefore, the BMPs cover state-of-the-art developments, including unit process approaches and specific control alternatives. The major emphasis is on source storage alternatives.

The presentation of combined sewer overflow controls describes the most promising full-scale operating facilities from a systems approach. Specific information updating costs and effectiveness of unit process alternatives provides a basis for developing an areawide control strategy that combines various control measures. Again, the presentation focuses on storage as the most important element of a systems approach.

### BEST MANAGEMENT PRACTICES

BMPs control urban runoff volumes and pollutant loads at the source or in upland areas rather than at the discharge point to the receiving water, where more conventional treatment methods are used. BMPs, such as street cleaning and sewer cleaning, can also be used to control source loads in combined sewered areas.

BMPs are either nonstructural or low structural controls. Nonstructural controls involve planning, modifying maintenance practices, and controlling development or natural land conditions to reduce runoff or pollutant potential. Low structural controls use natural land features with minor modifications or small, simple structures to control volume and pollutant generation at the source.

BMPs can control runoff from developing and developed areas. Planning is generally limited to new development, but maintenance practices and low structural controls apply to both.

Source controls often have multiple benefits. In addition to providing flood and erosion protection, the controls can improve the quality of receiving water. The flood and erosion controls developed by the Soil Conservation Service are the most common, and many of the controls and concepts used for urban stormwater management have been developed from Soil Conservation Service designs and criteria.

### Soil Conservation Service

The Department of Agriculture Soil Conservation Service (SCS) gives technical assistance to individuals, organizations, and local governments to control soil loss and provide water resource management in urban and rural areas [1]. The types of controls the SCS has promoted reduce erosion/sediment, flow, and flooding problems. These controls often have another benefit, stormwater pollution control.

The SCS assists many local resource conservation districts and county environmental protection departments. It provides a tangible benefit to stormwater control programs by sharing and using technology on soils, soil loss, and water resource management. In Montgomery County, Maryland, the local SCS reviews drainage plans and control measures as part of the county's permit issuing sequence for new development. The SCS also has compiled and produced a cost study for source detention ponds and has helped develop onsite stormwater management policy and guidelines [2].

In Middlesex County, Connecticut, the SCS assists in an environmental review of proposed development and control facilities. It has provided input on the distribution, quantity, and quality of the natural resources of the sites and surrounding areas and their ability to support the proposed development. The SCS also has prepared a handbook on the erosion and sediment controls used in Connecticut [3]. The handbook discusses how to design structural and nonstructural controls and the principles of erosion, sedimentation, hydrology, site selection, and planning.

In addition to local assistance, the SCS has prepared a design methodology for evaluating runoff volumes and flows from different land uses and soil types [4]. This manual is useful in estimating and evaluating the effects of land use changes and structural measures on hydraulic and hydrologic parameters.

### Selection of BMP Strategies

The selection of a BMP or a combination of BMPs depends on the area's characteristics and the problem to be corrected. For example, street cleaning may be considered for the control of source pollutant loads generated on urban streets. But, detention or detention/treatment may also be required to control peak flow volumes or treat urban runoff flows that contain pollutant loads from offstreet areas. The type of detention selected (onsite, offsite, inline, offline) depends on the availability of land, the type of drainage system (sewers or open channels), and the land use. Other considerations needing evaluation may include the potential downstream effects resulting from upstream BMP implementation, such as altered timing of stored flow volumes.

The costs of BMPs should also affect the control program selected. Nonstructural solutions usually cost less and may have levels of pollution control similar to low structural solutions. The costs of land use planning for new developments may be as low as about \$490/ha (\$200/acre) [5]. Street cleaning in impervious areas like parking lots can cost as much as \$3,200/ha (\$1,300/acre) [6], and studies in San Jose, California, indicate that street cleaning can cost \$8.70 to \$9.30/curb-km (\$14 to \$15/curb-mile) [7]. For the more common types of low structural controls, the construction costs range from about \$2,500 to \$9,900/ha (\$1,000 to \$4,000/acre) as shown in Table 2.

Table 2. RANGE OF CONSTRUCTION COSTS FOR LOW STRUCTURAL SOURCE CONTROLS

| | Cost, \$/acre[a] | |
|---|---|---|
| Source control measure | Average | Range |
| Detention/sedimentation basins | 2,200 | 1,000-3,800 |
| Percolation ponds | 2,600 | 1,800-3,500 |
| Swales/underdrains | 2,300 | 1,900-2,600 |
| Erosion controls | -- | 1,500-38,000[b] |

a. Per acre of gross controlled area.

b. Per acre of disturbed area.

The effectiveness of BMPs can range from total pollutant capture for systems capturing and percolating the total runoff volume from a development, to pollutant trap efficiencies for detention basins as high as 90%. The trap efficiencies depend a great deal on the design criteria, i.e., multiple storm frequency design to provide adequate detention volume for different size storms, and use of large permanent pools to hold pollutants [8]. Large permanent pools provide continued settling and oxidation of organic material after storms are over.

## Nonstructural Controls

Nonstructural controls can be the least costly BMP for the control of urban runoff since they usually involve modifying maintenance practices or enforcing ordinances that control development and planning practices. The effectiveness of these measures, however, is not well documented, and their benefits are therefore determined more by an intuitive judgment.

The most promising solutions for nonstructural control are land use planning, street cleaning, and establishing a public agency organized to adopt and enforce ordinances, conduct areawide control projects, and levy a stable and equitable source of funding. These solutions are not meant to be applied exclusively, but rather represent alternative elements available for use in an areawide control approach depending on local characteristics.

### Land Use Planning--

Land use planning should be used as a first step to prevent urban runoff problems from occurring during development because urbanization upsets the natural hydrologic and ecologic balance of a watershed. The degree of change, beneficial or detrimental, depends on the mix, location, and distribution of the proposed land use activities.

The potential pollutant concentrations in runoff from different land use activities in the Lake Tahoe area are compared in Table 3. Generally, increasing density and intensity of use creates higher pollutant concentrations. Commercial land uses have the highest pollutant potential, particularly for lead and zinc; the pollutant concentrations can be over five times the concentration from rural or undeveloped land.

Table 3. COMPARISON OF POLLUTANT CONCENTRATIONS IN RUNOFF FROM SEVERAL LAND USES [9]

| | Pollutant concentration, mg/L | | | |
|---|---|---|---|---|
| Land use | SS | Total N | Total P | Oil and grease |
| Rural/undeveloped | 50 | 0.2 | 0.1 | 0.6 |
| Low density residential | 600 | 1.2 | 0.7 | 0.8 |
| High density residential | 250 | 0.7 | 0.8 | 20.0 |
| Commercial | 770 | 1.7 | 1.3 | 33.0 |

Storm pollutant load yields from the different land uses show similar trends. In the Washington, D.C., area, nitrogen and phosphorus storm yields from high density residential and commercial land uses are over 10 times the load from rural or undeveloped areas [10].

The goal of land use planning as a source control is to limit land use activities with high pollutant yields to areas of the development that can support the intended activity and protect the receiving waters. Planning therefore may involve limiting development on steep erodible soils and in flood plains and retaining undisturbed areas and using them as buffers to development.

The approach to land use planning should have the following four elements to balance the economic impacts of a planned development with the potential environment benefits:

- Physical analysis of the site: identification of the environment--soils, geology, slopes, drainage, access, and types of development suitable for the site.
- Market analysis: identification of the public needs and interests in the types of activities and facilities to be developed.
- Economic analysis: examination of the costs and profitability of developing facilities on the land available.
- Regulatory requirements: coordination with local, regional, and state agencies to conform with environmental legislation and ordinances.

A comparison of a well-planned and a poorly planned development in the Lake Tahoe area showed a 100-fold reduction in sediment yield from the site using land use planning criteria [5]. This difference was also evident in the stream benthic sampling results. Below the poorly planned development, the macroinvertebrate community showed sharp decreases in density, number of families, and diversity, as shown in Table 4.

Table 4. MACROINVERTEBRATE SAMPLING RESULTS ABOVE AND BELOW A POORLY PLANNED DEVELOPMENT [5]

| Date of sampling | Density, No./m$^2$ | | Number of species | | Species diversity index[a] | |
|---|---|---|---|---|---|---|
| | Above | Below | Above | Below | Above | Below |
| Jul 1975 | 1,542 | 267 | 20 | 9 | 2.50 | 1.91 |
| Dec 1975 | 1,321 | 277 | 19 | 12 | 2.25 | 2.08 |
| Jun 1976 | 2,125 | 1,652 | 14 | 14 | 2.15 | 1.85 |
| Oct 1976 | 1,560 | 19 | 14 | 4 | 2.21 | 1.35 |

a. A measure of the relationship between the number of species and the total biologic community population by the Shannon-Weaver index: diversity = $-\Sigma(Ni/N)\ \ln\ (Ni/N)$, where Ni = number of species and N = total community population.

Corresponding sampling results below the well-planned development, in most cases, exceeded the quality of the above stream sampling at the poorly planned development.

In the planned community at The Woodlands, Texas, sampling results showed, in most cases, that the runoff water quality, particularly for nitrogen and phosphorus loads, was better than the runoff quality sampled at two urban areas near Houston, as shown in Table 5.

Table 5. COMPARISON OF URBAN RUNOFF POLLUTANT LOADING FROM THE WOODLANDS AND TWO FULLY DEVELOPED URBAN AREAS [11]

| | | Rank (Decreasing pollution ⟶) | | | |
|---|---|---|---|---|---|
| Pollutant | | 1 | 2 | 3 | 4 |
| Suspended solids | Area | P-30 | HB | WB | P-10 |
| | Mean lb/acre-in. | 43 | 38 | 14 | 8.2 |
| | Confidence limits[a] | ±18 | ±5 | ±74 | ±2 |
| COD | Area | HB | P-10 | P-30 | WB |
| | Mean lb/acre-in. | 19 | 14 | 13 | 9.5 |
| | Confidence limits | ±7 | ±1 | ±1 | ±24 |
| Soluble COD | Area | P-10 | P-30 | HB | WB |
| | Mean lb/acre-in. | 10 | 9 | 4.4 | 4.1 |
| | Confidence limits | ±1 | ±1 | ±2 | ±28 |
| Total phosphorus | Area | HB | WB | P-30 | P-10 |
| | Mean lb/acre-in. | 0.28 | 0.24 | 0.021 | 0.014 |
| | Confidence limits | ±0.12 | ±0.66 | ±0.007 | ±0.003 |
| Kjeldahl nitrogen | Area | HB | WB | P-30 | P-10 |
| | Mean lb/acre-in. | 0.95 | 0.40 | 0.30 | 0.28 |
| | Confidence limits | ±0.10 | ±1.10 | ±0.06 | ±0.10 |
| $NO_3$ | Area | WB | HB | P-30 | P-10 |
| | Mean lb/acre-in. | 0.088 | 0.087 | 0.020 | 0.012 |
| | Confidence limits | ±0.032 | ±0.013 | ±0.018 | ±0.008 |

Note: P-10, P-30 - The Woodlands
HB - Hunting Bayou (Houston)
WB - Westbury (Houston)

a. 95% confidence limits.

The cost of land use planning associated with environmental protection is extremely difficult to separate and assess. For the well-developed site at Lake Tahoe, the entire cost of the planning was attributed to runoff pollution control, which amounted to about $1,900/ha ($220/acre).

Street Cleaning--

Street cleaning can effectively control street-originating pollutants (heavy metals), and is moderately effective in controlling oil and grease, floatables, and salts. It is less effective in controlling sediment, nutrients, and oxygen-demanding matter [7]. Therefore, street cleaning should be selected and tailored to mitigate specific problems and may be required as one of a combination of measures to provide areawide runoff control.

Street cleaning is practiced in most areas of the country; however, it has not been used as a BMP. Modification to or the development of a street cleaning program can be a most promising source control.

The recommended elements in designing an effective street cleaning source control program are as follows [7]:

1. Determine an allowable street surface residual loading from the city's street cleaning objectives. These objectives are determined by environmental, aesthetic, safety, and public relations requirements to meet urban runoff load allocations.

2. Measure or estimate the long-term average particulate accumulation rate on street surfaces. This will vary with the street surface and the cleaning frequency.

3. Determine the maximum allowable effective days of accumulation and then determine required combinations of cleaning interval and cleaning efficiency.

The area characteristics (street surface conditions and accumulation rates) and the street cleaning program (number of passes and cleaning intervals) are more important for effective cleaning than the type of cleaning equipment [7]. The cleaning effectiveness, as a percentage of the initial solids loading removed, ranged from about 40 to 60% for asphalt streets in good condition and from 0 to about 20% for streets with oil and screens surfaces. The effectiveness in terms of the cleaning interval on an annual basis is shown in Figure 1. The average removals for oil and screens and asphalt streets (both in good and poor conditions) in the San Jose study are summarized in Table 6.

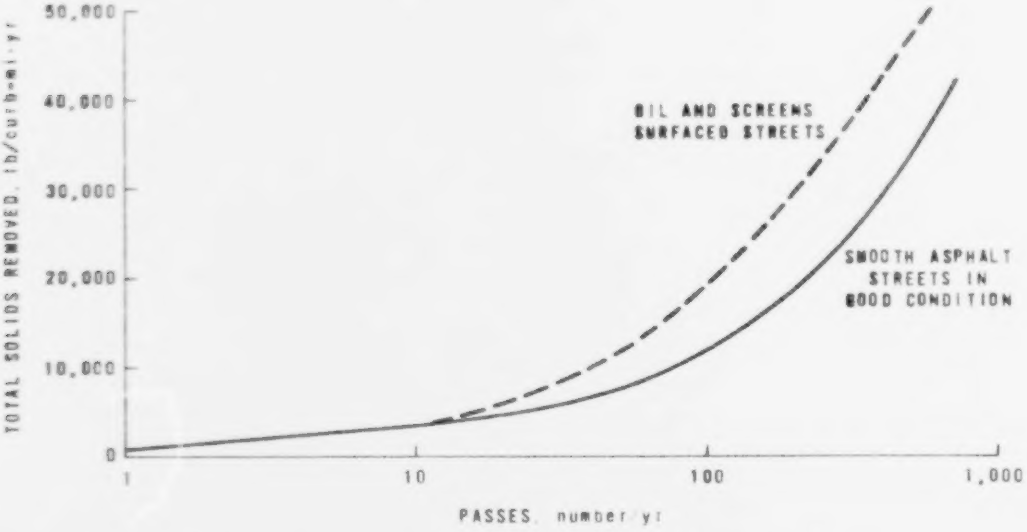


Figure 1. Variation of annual solids removal with number of equipment passes [7].

## Table 6. POLLUTANT REMOVAL EFFECTIVENESS ON DIFFERENT STREET SURFACES [7]

| | Removal, %[a] | |
|---|---|---|
| Parameter | Oil and screens | Asphalt |
| Total solids | 9 | 30-40 |
| COD | 9 | 30-45 |
| Kjeldahl nitrogen | 6 | 30-45 |
| Orthophosphate | 7 | 30-40 |
| Lead | 5 | 30-40 |
| Zinc | 12 | 30-45 |
| Chromium | 9 | 30-50 |
| Cadmium | 8 | 30-50 |

a. Includes streets in both good and poor condition in residential and commercial areas.

Detailed annual costs for street cleaning and all support activities in San Jose during 1976-1977 are presented in Table 7. The units costs are \$9.25/curb-km (\$14.88/curb-mi) [7]. A generalized cost curve based on the number of passes and the condition of the street is shown in Figure 2.

### Local Approach--

Many local agencies have limited budgets and manpower to implement BMP programs beyond the plan review, permit issuing, and construction inspection stages. Since most BMP facilities are implemented at the local level, information on the use and effectiveness of areawide controls has lagged behind the information on federally funded combined sewer overflow control measures.

The City of Bellevue, Washington, has successfully operated a separately funded stormwater utility that may promote the use of areawide controls. In addition, the utility monitors and enforces controls on privately owned developments to ensure compatibility with the city's overall water quality goals. This approach is one of the most promising organizational techniques for controlling urban runoff.

*Public Stormwater Utility*--The advantages of a utility approach to stormwater control are a stable source of funding and a centralized technical and operational staff that deals only with water resource related problems. Charges are made on all property, developed and undeveloped, based on their contribution to the runoff problem. The organization of the utility places utility inspectors, plan review engineers, water quality technicians, and maintenance personnel under a single operation, rather than drawing on personnel from a public works department, where stormwater control may have lower priorities.

### Table 7. ANNUAL STREET CLEANING COSTS AND LABOR REQUIREMENTS, SAN JOSE [7]

| | Costs | | Labor | |
|---|---|---|---|---|
| Item | $/yr[a] | Percentage of total | Man-d/yr | Percentage of total |
| Maintenance supplies[b] | 97,000 | 12 | -- | -- |
| Operation supplies[c] | 30,000 | 3 | -- | -- |
| Debris transfer and disposal[d] | 67,000 | 8 | 780 | 13 |
| Equipment depreciation | 32,000 | 3 | -- | -- |
| Labor[e] | | | | |
| Sweeper operators | 338,000 | 41 | 3,400 | 56 |
| Maintenance personnel | 183,000 | 23 | 1,200 | 20 |
| Supervisors | 83,000 | 10 | 650 | 11 |
| Total | 830,000 | 100% | 6,030 | 100% |

a. ENR 3000.

b. Includes broom replacements.

c. Tires, fuel, and oil.

d. Front-end loaders removed interim piles from streets, and dump trucks transported them (maximum 15 miles) to landfill.

e. Includes administration, warehouse, secretary, and overhead costs.

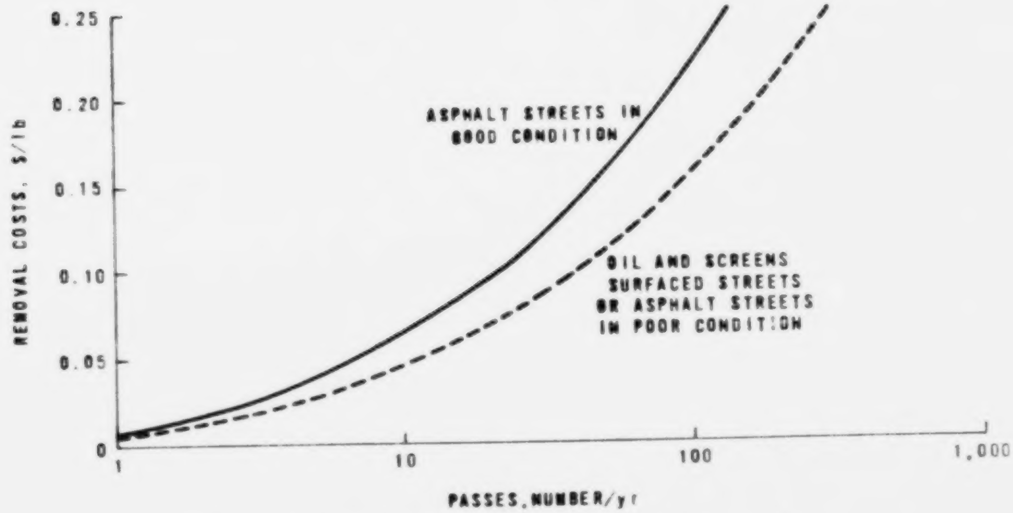


Figure 2. Variation of unit cleaning costs with number of passes [7].

Several problems, principally socioeconomic, can develop when organizing a separate stormwater utility. The public may not easily understand the services that a stormwater utility can provide. The public can relate to the services for water, sewer, and garbage collection; the benefits of stormwater control, however, are not readily apparent. This can lead to potential problems in developing a rate structure acceptable and equitable to the public.

Other Organizational Approaches--Other approaches to stormwater control have been set up and used as management and funding agencies to carry out stormwater policy and projects. These include (1) using a county or local government as the managing agency supported by the property tax base, or (2) creating a special improvement district with authority to levy a property tax for stormwater projects.

Davis County, Utah, evaluated these two alternatives and chose to make the county government the managing agency because, under Utah law, the county has authority over stream channels in both incorporated and unincorporated areas, can levy property taxes, and would, with voter approval, have authority to issue bonds for capital improvements. In addition, the county government can use planning and zoning powers to pass ordinances regulating development for urban runoff control [12].

The special improvement district, while having the operational advantage of a separate governmental agency exclusively for runoff control, would not have authority for land use planning and regulating development. Another concern was that the creation of a new and separate agency may be a difficult approach, politically.

## Low Structural Controls

Low structural controls require few structural facilities or modifications other than berms, earthwork, outlet devices, or minor modifications to the land surface features to control urban runoff. They are applied at the source or in upland areas of a watershed, and control runoff in new developments or mitigate existing problems in developed areas.

The most common low structural control in the country is storage. Storage attenuates peak runoff flows, treats runoff (detention/sedimentation), or totally contains the flow in combination with another treatment process (retention/percolation).

Most of the storage facilities are usually constructed onsite, where the runoff or pollutant problems begin and are wet or dry ponds. Approximately 42% of all the low structural controls in Montgomery County, Maryland, are storage ponds used to control areas of 0.4 to 2.0 ha (1 to 5 acres) [13, 14]. For larger control areas, up to 200 ha (500 acres), storage ponds are used almost exclusively and are constructed offsite to control entire developments or small drainage basins [14].

Erosion controls and other types of storage can control specific stormwater problems, where the site or the condition of the soils make these solutions particularly attractive. The low structural controls selected should, therefore, be based on design requirements or regulations, cost, effectiveness, and conditions of the site.

Storage/Rate Control--

Storage controls the peak runoff rate from sites that have become more impervious because of development. By controlling the rate at which stored runoff is released, potential downstream damage including flooding, streambank erosion, and damage to vegetation can be lessened. Estimates of annual damages from storm flows in a 1,170 ha (2,900 acre) watershed in Montgomery County, Maryland, exceeded $400,000, or about $370/ha ($150/acre) of gross watershed area [15].

The peak flow reduction is a function of the maximum usable storage volume and the release rate of the outlet control. Storage basins designed to control a peak flowrate from a single design storm frequency, for example a design based on a 2 yr storm, can reduce the peak flow over 90% on the average for that storm. This efficiency, however, drops significantly (50 to 60% or less) for storms that exceed the design storm [16]. The effectiveness of storage may be increased by using outlet controls to detain flows from several design storms so that peak flows from storms up to the 100 yr storms could be reduced by 70 to 80% [17].

Rooftop and parking lot storage facilities have been used for small onsite applications to control flows from highly impervious areas. Rooftop storage detains water on roofs, releasing the volume through roof drains. Parking lot storage is created by constructing depressions in the parking lot to store excess runoff. The released flow is controlled by limiting the drain capacity from the lot. Other methods of onsite storage include underground storage vaults or oversized underground pipes, but they often cost more.

Rooftop and parking lot storage are highly feasible alternatives for detaining urban runoff from small sites in developed areas, where land may be unavailable for the construction of detention basins. Since most roofs are flat, watertight, and structurally designed to withstand loads greater than ponded water, it adds very little to the cost of a building to provide for rooftop storage. Similarly, parking lots can easily be designed to store water and still function as parking lots with little inconvenience to people.

Storage/Treatment--

Storage can control stormwater pollution by providing treatment or total containment (retention). Retention usually involves such processes as infiltration. Storage can be used onsite or offsite. Storage can treat runoff in the following ways:

> Facilities that totally retain storm runoff will have no surface water pollution loading from that site. Infiltrating water can,

however, pollute the groundwater, and an analysis should consider the filtering effect of the soil, depth to groundwater, and groundwater use.

- The flow-controlling properties of detention storage prevent rapid changes in flow regime, which is a form of pollution. Increased stream velocity suspends silt, stirs up bottom deposits, and disturbs the habitats of organisms living in pools or in contact with stream banks.

- A receiving water has a natural ability to assimilate and neutralize pollutants and is degraded only when this ability is overwhelmed by pollutant loading. Delaying the release of runoff and extending the pollutant loading over a period of time helps the stream assimilate the contaminants.

- Onsite storage keeps rainwater from flowing over urban surfaces that are potential sources of pollution. Runoff forced to travel over urban streets picks up a large quantity of street surface contaminants.

- The decreased velocity of storm runoff caused by detaining peak volumes means less channel erosion in natural streams and earthen conduits. The lower velocities also mean a lower sediment-carrying capacity.

- The detention of stormwater in a pond for a period of time results in settling and decreases the particulate loading of the outflow. Biological oxidation of organic materials also occurs in permanent pool detention ponds. This can include aerobic and anaerobic (benthos) treatment or stabilization.

Onsite Detention/Retention Basins--Onsite detention uses simple ponding techniques in open areas to accumulate stormwater. The basic design elements are a contained area that allows the stormwater to pond and a release structure that controls the rate at which the runoff enters the downstream drainage system.

Onsite detention ponds can be instream or offstream facilities. Instream facilities are flow-through controls that are a part of the drainage system. With instream storage, a weir, check dam, restricted pipe, or other outlet control uses the existing storage capacity in the drainageway. Offstream detention facilities are usually natural or excavated depressions or swales into which high flow volumes are diverted during storms and later drained back into the drainage system.

Onsite detention is used for new, small, privately owned developments or individual sites, such as industrial complexes, because it is simple to construct and offers one of the lowest cost alternatives for stormwater control. Another type of onsite detention is the temporary sediment pond used during construction to prevent sediment from disturbed or unvegetated soils from entering receiving waters. Most local authorities throughout the country require similar temporary measures as part of their sediment control regulations and ordinances.

Several commercial, industrial, and residential developments have used retention basins to capture the entire runoff volume on the site and dispose of the water by percolation. These measures are most appropriate for small sites, usually less than 2 ha (5 acres) of impervious area. The application of retention/percolation systems greatly depends on the percolation capacity of the soil. The costs of these systems are equivalent to or higher than the costs of onsite detention/sedimentation basins, but these systems can reduce flow and pollution load by 100%.

A 3 ha (7.5 acre) light industrial site in Connecticut uses a series of dry wells to capture and percolate the runoff from the building and parking lot at an estimated cost of $16,000/ha ($6,600/acre) of impervious surface. Another industrial development using detention/sedimentation costs $10,800/ha ($4,400/acre).

Offsite Detention Basins--Offsite detention can control several developments or small watersheds and has been gaining in acceptance. While the construction costs of these larger basins are higher, the cost per unit area controlled is lower and the level of control is often increased by designing the facilities to control storms of up to 100 yr return frequencies.

In many areas of the country, developers contribute funds to regional control facilities in lieu of providing onsite controls. Maintenance costs can be reduced by placing the maintenance responsibility on a public authority for one large facility, rather than operating an equivalent number of smaller onsite basins on the same area. In addition, maintenance operations on large publicly owned facilities have a better chance of being properly carried out; the required maintenance on small, privately owned facilities is often forgotten after they are constructed. The possibility exists, however, for increased conveyance or drainage system costs to the offsite control facility.

Offsite detention basins are usually constructed in a drainage system. In Montgomery County, large dams are built across streams below development, and pool areas are formed either by the existing topography or by excavating and reshaping the topography [17, 18]. The outlets are engineered to control several storm frequencies and most maintain permanent pools. The usable storage volume is, therefore, provided above the permanent pool elevation.

In Bellevue, Washington, the city plans to construct offsite detention basins within the drainage system and has adopted local ordinances that require new development to control onsite runoff during and after construction. The offsite storage facilities will control runoff in the entire city, including existing developments. The types of offsite basins range from simple drainage channel or culvert modifications to create storage, costing $0.35 to $3.50/m$^3$ ($0.01 to $0.10/ft$^3$) of created usable storage, to more expensive facilities where major excavation and outlets are required, costing $7.00 to $23.00/m$^3$ ($0.30 to $0.65/ft$^3$) of storage [19].

Offsite regional control of existing development and existing runoff and pollutant problems is one of the greatest values or advantages of this alternative. Constructing onsite controls on existing developments is prohibitively expensive and might encounter public opposition.

Construction Costs--

The cost of constructing onsite or offsite storage facilities depends on the natural features of the proposed site. Storage basins created from natural depressions, or in wetlands requiring only minor control structures and limited earthwork, are up to 10 times less expensive than basins that needed major excavation, as shown in Figure 3. The cost curves, developed from planned offsite detention basins in Bellevue, show distinct scales of economy in unit construction costs for basins with storage capacities ranging from 7,000 $m^3$ (250,000 $ft^3$) to 113,000 $m^3$ (4,000,000 $ft^3$) [19].

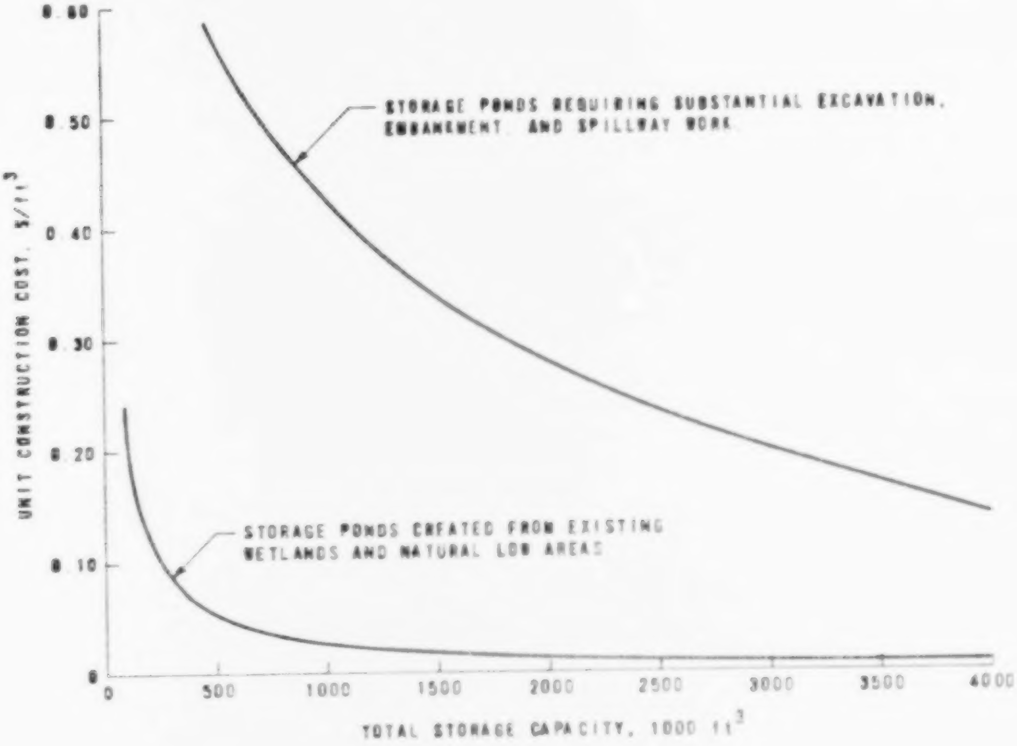


Figure 3. Storage pond construction costs, ENR 3000.

The cost of the land for offsite storage facilities should also be considered in selecting a storage alternative, particularly if the land is not publicly owned. In Bellevue, an estimated 86 ha (212 acres) would be needed to construct detention basins in one watershed at an average cost of $54,000/ha ($22,000/acre) [19].

A cost curve for estimating the construction cost of detention ponds with storage volumes from 28 to 28,300 $m^3$ (1,000 to 1,000,000 $ft^3$) was developed by the SCS using actual construction costs in Montgomery County, Maryland, and is shown in Figure 4 [14]. These costs are representative of small onsite detention ponds and are about 2 to 3 times the cost of offsite detention ponds created from existing wetlands and natural depressions.

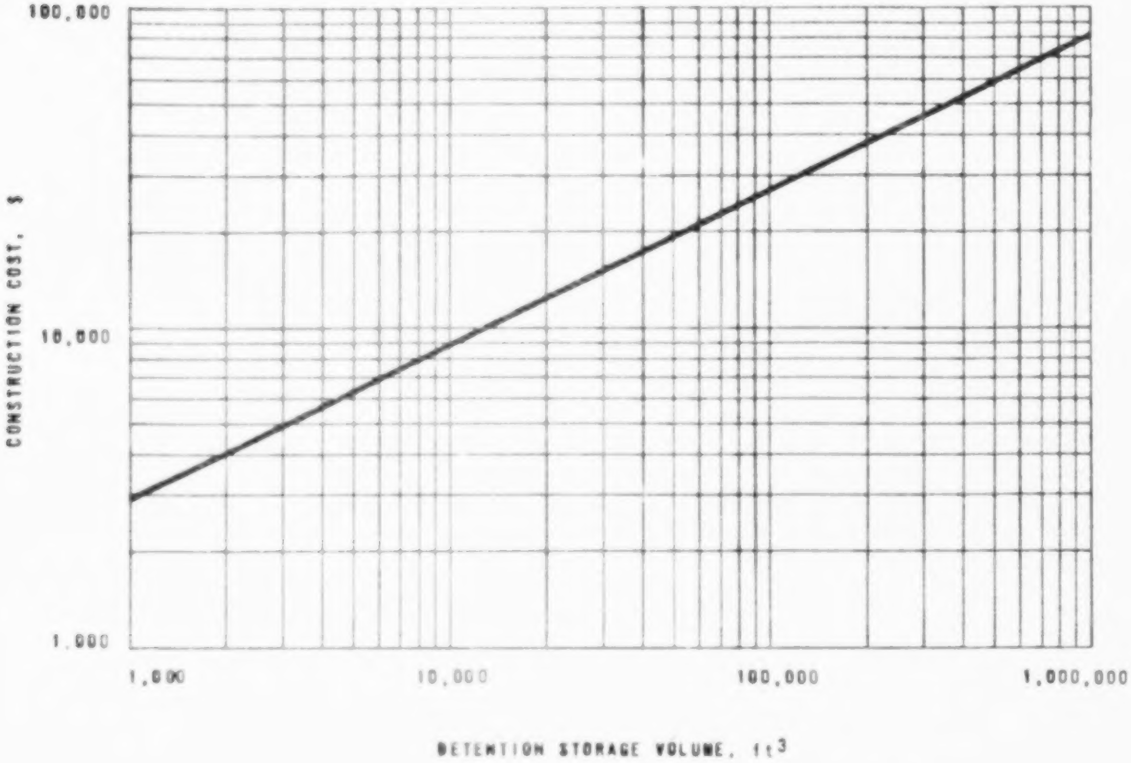


Figure 4. Onsite stormwater detention pond costs, ENR 3000 [14].

Estimated costs of onsite retention/percolation facilities range from \$10.60 to \$15.90/$m^3$ (\$0.30 to \$0.45/$ft^3$) of storage capacity provided. These costs, developed for the Florida area, apply to small facilities controlling up to about 12 ha (30 acres) of contributing watershed area [20, 21]. The size of the basins ranges from 500 to 4,500 $m^3$ (18,000 to 160,000 $ft^3$) with an average depth of 1.5 m (5 ft).

Operation and Maintenance Costs--

Information on the annual operation and maintenance cost of source detention facilities is limited because many are small, privately owned onsite basins; the accounting of costs has been incomplete; or the facilities have not been maintained after their construction. Estimates of operation and maintenance costs from several facilities, however, indicate the annual costs for low structural source detention. In Florida, the annual operation and maintenance costs for retention/percolation systems ranged from about \$740 to \$1,500/ha·yr (\$300 to \$600/acre·yr) of service area.

Operating and maintaining large offsite facilities can result in annual cost savings over the use of many small onsite facilities. For example, the estimated operation and maintenance cost for six large offsite detention basins controlling a 1,170 ha (2,900 acre) area in Montgomery County, Maryland, is \$84,000, or approximately \$72/ha·yr (\$30/acre·yr) [15]. This is at least 10 times less than the costs for the small onsite facilities in Florida.

### Design Guidelines--

Design guidelines for low structural controls vary from region to region and can also vary within a local jurisdiction depending on the problem being solved, i.e., stormwater quality control or flood control. Most local jurisdications, however, have ordinances that require both urban runoff controls for all new development and designs that limit runoff from the development to predevelopment rates.

Storm Frequency--A minimum design requirement for control of a 2 yr storm flow is common in areas that have comprehensive ongoing control programs. Outlet controls on large offsite detention facilities can handle up to the 100 yr storm flow [17]. These large facilities protect downstream receiving waters from peak flowrates more than would an offsite facility designed just for a 2 yr storm. The impact of different storm frequency designs on flow control performance is shown in Table 8.

Table 8. RESPONSE OF PEAK STORM FLOW THROUGH A DETENTION SYSTEM DESIGNED FOR SPECIFIC RETURN PERIODS [22]

| | Storm flow return period, yr | | |
|---|---|---|---|
| Level of control/design | 2 | 10 | 100 |
| Predevelopment flow, $ft^3/s$ | 9 | 29 | 77 |
| Post-development flow through design control, $ft^3/s$ | | | |
| 2 yr storm released at 2 yr predevelopment rate | 9 | 22 | 42 |
| 10 yr storm released at 10 yr predevelopment rate | 14 | 29 | 57 |
| 10 yr storm released at 2 yr predevelopment rate | 6 | 9 | 27 |
| 100 yr storm released at 100 yr predevelopment rate | 15 | 33 | 77 |

Design requirements specifying maximum runoff or release rates from storage have also been used [23]. Typical design requirements for controlling urban runoff are summarized in Table 9.

Table 9. TYPICAL DESIGN REQUIREMENTS FOR URBAN RUNOFF CONTROL FACILITIES

| Location | Design requirement |
|---|---|
| Bellevue, Washington [23] | Limits runoff to predevelopment rates, maximum runoff or storage release rate at 0.2 $ft^3$/acre·s, storage requirements based on 100 yr storm. |
| Montgomery County, Maryland [14, 15, 17] | Limits runoff to predevelopment rates, 2 yr storm frequency design. In practice, regional offsite detention facilities also provide controls for the 10 and the 100 yr storm frequencies. |
| Orange County, Florida [24] | Detains first 1 in. of runoff and release at specified rates for flood protection. |
| Middlesex County, Maryland [25] | Has no specific design criteria for control facilities, but requires controls during and after construction. Drainage facilities are designed for 25 yr storm. In practice facilities have been designed for the 50 and 100 yr storm. |
| Boulder, Colorado [27] | Limits runoff to predevelopment rates. Has no specific design storm requirements other than that drainageways must accommodate flows from the 100 yr storm. |

Wet Versus Dry Ponds--Selecting a wet or a dry pond design depends on many factors--design purpose (pollution control/flow control), public safety, maintenance difficulties, land area requirements, and appearance. In addition to functional considerations, many of these factors are socioeconomic considerations that may significantly influence selection.

A wet pond design controls both flow and pollution by maintaining a permanent pool and a storage volume above the pool level. The permanent pool in the wet pond traps pollutants. The pool volume maintains sufficient detention times so that the pond can treat pollutants that would otherwise be drained from a dry pond to the receiving water. In a dry pond, settled pollutants from previous storms are resuspended and washed out of the basin with the next storm flow.

The safety features of a wet pond depend on the potential secondary use of the facility; a dual-purpose recreational lake cannot be fenced to prevent access. Typical safety features of a wet pond are shallow bank slopes, fences, and outlet guards.

Maintenance of wet and dry ponds is often inadequate or deficient and should therefore be considered or provided for during design. Maintenance measures can include removing debris or sediment, landscaping, and maintaining the outlet structure. Inadequate maintenance can adversely affect the operation, efficiency, and flow characteristics of the stormwater control facility. The two major problems are sediment deposition and vegetation growth in the emergency spillway [22].

Both mosquito and algae problems can be eliminated in dry basins by ensuring that the areas dry out completely between uses. For permanent pool ponds, these problems are more difficult to control. Mosquito breeding can be upset by removing the grass at the shoreline, varying the pond water depth every few days, or stocking the ponds with larvae-eating fish.

The best way to overcome objections to setting land aside for a detention pond is to recognize that the area can be an asset as open space. Housing near greenbelts and pond areas usually has a higher market value if the open space is aesthetically designed.

Dry detention ponds are most presentable when a grass cover is kept on the basin slopes and floor. Grasses can be grown that will withstand periodic flooding. If retention basins contain water for long periods or need to be vegetation-free for better infiltration, appearance objections may be overcome by sight barriers such as trees.

In most cases, a public agency should own and operate dry and wet ponds. Public agencies usually have more equipment, manpower, and expertise available than homeowner associations and developers.

Design Approach--The approach for designing low structural controls can range from graphical techniques to complex mathematical models. The stormwater design approaches and important design criteria are summarized in Table 10.

Table 10. HYDROLOGIC DESIGN APPROACHES
FOR STORMWATER DETENTION FACILITIES [22]

| | Design approach | | | | |
|---|---|---|---|---|---|
| | | | | Conceptual models | |
| Evaluation criteria | Graphical | Empirical | Unit-hydrograph | Simple | Complex |
| Design elements | | | | | |
| Multiple return period | No | Maybe | Yes | Yes | Yes |
| Storm duration | No | Maybe | Yes | Yes | Yes |
| Maintenance | No | No | No | Maybe | Yes |
| Soil characteristics | No | No | Maybe | Yes | Yes |
| Downstream effects | No | No | Maybe | Yes | Yes |
| Accuracy | Low | Low | Medium | High | High |
| Training requirements | Low | Low | Medium | Medium | High |
| Relative design cost | Low | Low | Medium | Medium | High |

Stormwater detention system design should incorporate intensity-duration-frequency concepts to account for important variables of urban runoff events that can affect the control's performance. The SCS method is an example of a simple conceptual model and is regarded as one of the most appropriate design approaches for stormwater facilities. Its principal features are a desktop approach that has high accuracy-medium training requirements and is capable of evaluating multiple return period storms, storm duration, and soil characteristics.

## Erosion Controls

Erosion controls limit the adverse impacts of uncontrolled stormwater runoff or correct existing erosion problems resulting from poorly planned development. They also can limit sediment and soil loss from construction sites. Erosion controls include temporary soil stabilization, permanent slope stabilization, runoff control, and revegetation.

### Temporary Soil Stabilization--

Temporary soil stabilization methods are used on disturbed slopes or areas to provide erosion control, dust control, mulch, or mulch protection. The effectiveness of these methods is short lived. The cost of these methods ranges from about $1,500 to $24,700/ha ($600 to $10,000/acre), as shown in Table 11.

### Permanent Slope Stabilization--

Permanent slope stabilization controls are mechanical methods that physically change the disturbed slope area or provide physical barriers to support the slope. The methods described in Table 12 do not provide mulch or surface protection to bare slopes and require temporary slope stabilization methods until permanent vegetation is established.

### Runoff Controls--

Runoff controls are used in construction areas and are used in addition to slope stabilization controls for increased effectiveness in mitigating erosion. Runoff controls include diversion dikes, interception trenches, pipe drops, chutes and flumes, sediment barriers, and berms. The unit costs of these controls are summarized in Table 13.

### Revegetation--

Vegetation provides the best long-term protection to sloped surfaces and is the ultimate goal of providing erosion controls to disturbed areas. The costs to revegetate disturbed areas can vary significantly depending on the slope; the need for slope stabilization, reshaping, seed bed preparation; and the method of application of seed or plantings. The costs range from about $2,500/ha ($1,000/acre) for seed application (hydroseeding with mulch) to over $67,000/ha ($27,000/acre) for plantings or rooted shrub cuttings. Seed and fertilizer application adds about $500/ha ($200/acre) to the cost of hydromulching.

## Table 11. DESCRIPTION AND COST OF TEMPORARY SOIL STABILIZATION[a] [5, 28]

| Method | Description | Total cost, \$/acre[b] |
|---|---|---|
| Jute matting | Mulch nets made of jute used for erosion control and protection of other mulches. | 9,100 |
| Matting in drainage channels | Application of jute matting or fiberglass roving for dust and erosion control in very small drainage channels with flow velocities less than 2 ft/s. | 9,500 |
| Plastic netting | Monolithic plastic cloth-like material used over mulch, straw, or hydromulch. | 4,800 |
| Wood excelsior matting | Mat of wood excelsior fibers bonded to a paper or plastic used for dust and erosion control. Flows under mat should be prevented. | 10,700 |
| Fiberglass roving | Matting of continuous strands of glass fibers and tacking agent. Used for dust and erosion control and as a mulch for seeded and unseeded areas. | 4,000 |
| Hydromulching | Mechanized rapid method for applying wood fiber mulch, and tacking agent with or without seeds to large areas. | 1,200[c] |
| Chemicals and tackifiers | Plastics, organic seeding additives, asphaltic tacking agents and other products used to tack fibers to slopes for erosion and dust control. | 600 |
| Wood chip application | Temporary mulch and surface protection using chips of wood. Used for dust and erosion control during construction and as a mulch around plantings. | 850 |
| Crushed gravel mulches | Application of gravel or crushed stone as a mulch to stabilize soils during construction, or for low-use dirt roads, driveways, and areas of light vehicular use. | 800 |
| Straw mulch | Application of staple straw as a protective cover over bare or seeded soil to reduce erosion and provide a mulch. Requires matting or other methods to hold it in place. | 680[d] |

a. ENR 3000.

b. Includes materials, labor, and equipment.

c. At 2.5 tons/acre.

d. At 2.0 tons/acre.

### Table 12. SUMMARY AND COSTS OF PERMANENT SLOPE STABILIZATION METHODS[a] [5, 28]

| Method | Description | Unit cost, $[b] |
|---|---|---|
| Rock retaining wall | A low gravity wall constructed of rock materials to provide an aesthetically attractive method for physical stabilizing a slope. | 27/lf[c] |
| Redwood retaining wall | A retaining wall constructed of redwood planking and posts to stabilize over steepen or unstable slopes. | 25/lf[d] |
| Gabions | Large, single- or multi-celled rectangular wire mesh boxes filled with rock and wired together for permanent slope or drainage stabilization and erosion control. | 21/lf[d] |
| Slope bottom bench | A gently sloping surface at the base of a steeper slope to retain eroded material. | 7/lf |
| Wattling | Bundles of live cuttings from willows to stabilize slopes and provide revegetation. Wattling reduces slope lengths for surface runoff, increases water retention, and provides additional organic matter. | 2.3/lf |
| Slope steeping | Continuous series of horizontal steps cut on the face of cut slopes to interrupt slope length and provide slope stabilization. | 570/acre |
| Slope serration | Construction of approximately 10 in. horizontal steps on the entire face of a cut slope to provide stabilization benches which can support vegetation. | 420/acre |

a. ENR 3000.

b. Includes materials, labor, and equipment.

c. 4 ft high wall.

d. 3 ft high wall.

Table 13. SUMMARY OF UNIT COSTS OF RUNOFF MANAGEMENT SOURCE CONTROLS[a] [28]

| Method | Unit cost, \$/lf[b] |
|---|---|
| Diversion dike | 4.24 |
| Runoff interception trench | 7.22 |
| Strawbale sediment barrier | 2.49 |
| Sandbag sediment barrier | 3.67 |
| Filter berm | 8.08 |
| Filter fence | 3.31 |
| Filter inlet | 2.30 |
| Siltation berm | 8.52 |

a. ENR 3000.

b. Includes materials, labor, and equipment.

## COMBINED SEWER OVERFLOW CONTROLS

Combined sewer overflows occur when the combination of high storm inflow volumes and sanitary sewage exceeds the capacity of the interceptor and treatment plant to transport and treat those flows, respectively. Control of combined sewer overflows can best be achieved by a systems approach that includes either storage or treatment or both. The system approach is a most promising solution to areawide control because it can (1) make use of the existing collection system; (2) combine control technologies (storage/treatment) to obtain a more cost-effective solution than the use of a single technology (either storage or treatment alone); and (3) integrate combined sewer overflow controls with dry-weather treatment facilities, providing opportunities for higher levels of control for stormwater pollutants.

Storage, the key to a systems approach, is the fundamental element in each of the three solution approaches. Storage of combined sewage, in addition to reducing peak flows, treats the combined sewage by allowing heavier solids in the detained flows to settle.

The treatment step in the systems approach is used to achieve direct pollutant removal to meet water quality goals. Treatment can include both physical and biological unit processes applied as end-of-pipe controls before discharge to a receiving water. Physical processes are the most widely used controls because of simplicity, ease of startup, and capability to handle transient flow and quality characteristics of combined flows. Some physical processes, such as the swirl concentrator/regulator, have been applied as upstream control devices with the concentrated flow going to a dry-weather treatment plant.

## Storage

Planning for storage in combined sewage control facilities should first consider the use of excess capacities in existing combined sewer trunklines and interceptors, then progress to offline storage or downstream end-of-pipe storage basins. Inline storage and routing costs the least and potentially has the greatest level of operational flexibility, not only for stormwater control, but also for dry-weather system operation. Offline storage facilities may be needed to provide the required level of flow control and to optimize the sizing and/or the use of downstream treatment facilities. This is particularly critical when a sanitary sewage treatment plant is integrated with the wet-weather control system.

### Inline Storage--

Inline storage is created by use of a flow restriction device either within the transport conduit or at an overflow point (regulators). Pumping stations in interceptors have also successfully controlled flows and used excess storage capacity [29]. Various levels of operational control are available for an inline storage system and can signficantly affect system effectiveness and flexibility. These considerations need to be evaluated in terms of cost and control requirements.

The cost of inline storage can range from as low as \$250/ha (\$100/acre) for simple inline flow restriction to over \$2,500/ha (\$1,000/acre) for a system that includes central computer controls, remote-controlled regulators and pumping stations, and total system surveillance and data collection. The Seattle system averages about \$182/m$^3$ (\$0.69/gal) of inline storage capacity.

In addition to benefits of increased combined sewer control, increased system control may provide benefits and cost incentives for dry-weather operations that are not readily recognized. For example, in Seattle, Washington, the computerized inline storage control system operates during dry-weather periods to provide:

- Continuous system surveillance of sanitary flows, sewage levels, and equipment operation.
- Continuous hard copy data base of system operation obtained by data logging equipment.
- Flexibility and capability to route and store sanitary flows, which allow maintenance crews to repair the system without bypassing untreated flows.
- Capacity to reduce diurnal or unexpected flow variations to the dry-weather treatment plant, potentially improving treatment performance.

The effectiveness of inline storage depends on the volume of excess usable capacity in the system and the level of control (simple local versus complex automatic or supervisory control). The amount of excess storage capacity directly affects the overflow volume reduction effectiveness. If the system configuration is such that inline storage cannot provide the required storage volume for the desired level of control, then offline impoundments in upstream or downstream areas may be required. In this situation, the inline storage system can be used to connect and control offline storage facilities in a common control network.

An example of the effects of various levels of system control is shown in Figure 5. The total physical excess storage volume in the system is the same for each level of control; however, by adding supervisory control of the system operation, the overflow volume was reduced by 60% over that achieved by local control of the regulators and pumping stations.

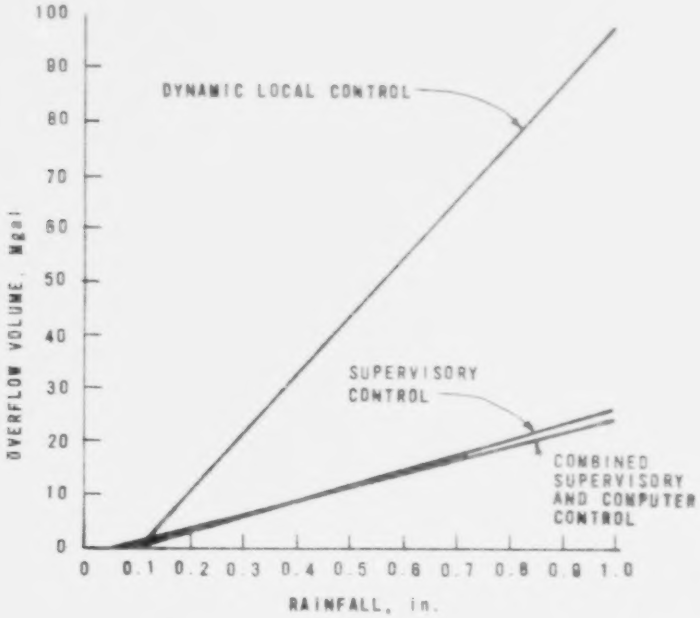


Figure 5. Comparison of Seattle's inline storage efficiency under three modes of operations.

The cost of adding the computer facilities and interfacing was about $5.7 million or approximately $940/ha ($380/acre) for the 6,080 ha (15,000 acre) combined sewer area [29]. The Seattle computer system represents the most sophisticated facilities used to control inline storage, to date. This level of control, however, applies to larger systems because the basic cost of similar computer facilities for a small system could significantly decrease the cost effectiveness of the overall system. The unit costs and the marginal benefits of the computer-controlled inline storage system become less attractive as the costs equal or exceed the unit costs for offline storage achieving a similar level of control.

Offline Storage--

Offline storage, alone or in combination with inline systems, increases the level of overflow control beyond the capacity of the inline system. Seattle plans to use offline storage to reduce overflows from approximately 40 to 10/yr [30]. The offline storage would be connected to the inline system. Saginaw, Michigan, uses offline and inline storage to control combined sewer overflows; and Mount Clemens, Michigan, uses storage as part of the wet-weather treatment system to reduce pumped flows entering the plant from 10.9 $m^3/s$ (250 Mgal/d) to an average of 0.2 $m^3/s$ (4 Mgal/d) before they enter the biological treatment lagoons [31].

In most applications of offline storage, a storage basin also operates as a sedimentation basin to handle flows that exceed the storage capacity. Most small storm volumes are totally contained in the basins, and after the storm, the combined sewage is released to the interceptor to flow to a dry-weather treatment plant as capacity becomes available. This type of operation is a most promising approach for both large and small systems, where the dry-weather treatment facilities permit processing of the extra flow and combined sewage pollutant load.

The cost of offline storage can range from about $105/$m^3$ ($0.40/gal) of storage capacity for simple earthen structures similar to those used to control urban runoff, to about $530/$m^3$ ($2.00/gal) for concrete storage sedimentation tanks such as those used in Saginaw, Michigan [32].

The effectiveness of offline storage can be addressed in several ways: (1) reduction of peak flows, (2) reduction of overflows, and (3) containment of pollutant loads. In most applications, all three considerations are incorporated into the design. Peak flow reduction provides a more uniform flow to downstream treatment processes, preventing possible process breakdown and flooding from transient flows and loadings. Similarly, depending on the volume of storage provided, downstream treatment capacity can be reduced, thus trading more expensive treatment costs for potentially cheaper storage costs. The proper balance of storage and treatment for the optimum cost-effective solution is one of the major design considerations in planning and evaluating alternative system approaches.

Storage can decrease the number of overflows occurring in a combined sewer system. Design considerations for sizing storage for overflow reduction should include regulations or water quality goals, statistical analysis of precipitation events, and storm duration. For example, a large storm volume with a long duration may not result in an overflow from storage, while a smaller storm volume with a short duration may. The effectiveness of the Saginaw, Michigan, storage/sedimentation basin, which was designed to contain approximately 1.3 cm (0.5 in.) of runoff from the contributing watershed, reduced the overflow volume about 60% and the overflow frequency about 73%. A summary of 11 storms, monitored during the summer of 1978, activating the storage/sedimentation facilities is presented in Table 14.

Table 14. SUMMARY OF STORAGE/SEDIMENTATION ACTIVATION EVENTS, SAGINAW, MICHIGAN

| Date | Average precipitation, in.[a] | Average intensity, in./h[b] | Volume pumped to basin, Mgal[c] | Overflow volume treated and discharged, Mgal |
|---|---|---|---|---|
| 5/12/78 | 0.21 | 0.11 | 2.62 | 0 |
| 5/30/78 | 0.68 | 0.14 | 4.19 | 0 |
| 6/12/78 | 0.44 | 0.19 | 3.60 | 0 |
| 7/21/78 | 0.42 | 0.15 | 1.08 | 0 |
| 8/16/78 | 0.50 | 0.29 | 3.70 | 0 |
| 8/19/78 | 0.52 | 0.15 | 5.12 | 1.60 |
| 9/12/78 | 0.48 | 0.11 | 3.68 | 0 |
| 9/13/78 | 1.35 | 0.17 | 16.79 | 13.27 |
| 9/17/78 | 0.32 | 0.13 | 0.90 | 0 |
| 9/20/78 | 0.90 | 0.28 | 9.66 | 6.14 |
| 9/27/78 | 0.39 | 0.11 | 0.89 | 0 |
| Total | 6.21 | | 52.23 | 21.01 |

a. Average of two rain gage measurements.

b. Average rainfall divided by average duration.

c. Starting on 8/19/78, volumes determined from pump operation logs. All previous values were estimated by water level observations.

Storage/sedimentation basins also store and trap combined sewage pollutant loads, in addition to treating flows that exceed the storage capacity. Pollutant loads from small storms that are totally captured are contained and released to the dry-weather treatment facilities for processing. On an annual load basis, excluding the efficiency of the dry-weather plant, the Saginaw storage/sedimentation basin removed approximately 89% of the suspended solids load and 81% of the BOD load.

### Treatment Processes

Treatment processes used for combined sewage range from simple physical processes to more operationally complex methods using biological processes such as treatment lagoons. The period of operation, operating costs, and treatment response times are important factors in evaluating and designing treatment alternatives.

The operation of combined sewage treatment facilities usually occurs only during storms. Therefore, the facilities and equipment may sit idle during dry-weather periods and may require special maintenance procedures to keep the equipment operational. This is especially critical if the mechanical equipment is in contact with the flows or in an enclosed corrosive environment.

Complex treatment processes usually have high operation and maintenance costs. This is particularly true if the process scheme uses chemicals.

Combined sewage treatment facilities are also subject to transient loading conditions. These can occur as first-flush loads, or varying loads, as flows from contributing watersheds arrive at the plant. The principal transient flow condition is a result of the storm characteristics. The ability of a treatment process to handle transient conditions is a key consideration in selecting a process. Process evaluations should also consider process effectiveness limits and process recovery rates back to stabilized operation.

Maintaining a continuous, relatively uniform feed is particularly important for biological systems and is a major problem in applying this process to stormwater control. Successful biological applications use dry-weather process units to treat wet-weather flows up to peak design loading rates consistent with the process being used. Treatment lagoons can be designed to provide a reliable biological process for combined sewage and can be maintained during dry weather.

A treatment approach should use the simplest, most mechanically free process that can achieve water quality goals. Physical primary treatment processes meeting these design and operational requirements are the most common type of treatment used throughout the country for both large and small systems. They usually have low hydraulic head requirements and sludge removal and washdown can be achieved hydraulically, thus having no mechanical equipment in the basin such as in Saginaw's storage/sedimentation basin [33].

The swirl concentrator/regulator is perhaps the simplest primary physical process. The swirl has no mechanical parts and can be constructed inline either for service as a regulator at an overflow point, where the concentrated underflow is returned to the interceptor, or as an end-of-pipe treatment device. At Lancaster, Pennsylvania, where a swirl concentration/regulator is being demonstrated, the concentrated combined sewage underflow enters a grit swirl and then receives further treatment at the dry-weather plant. The clear overflow is chlorinated before discharge to the receiving water.

Sedimentation--

Sedimentation treatment efficiency for combined sewage systems is a function of the influent solids concentration, particle settling velocities and the hydraulic loading rate in the basin. Process removal rates for the Saginaw facilities averaged 73% for suspended solids and 54% for BOD, based on monitoring for three storms that overflowed the sedimentation basin during the summer of 1978. The performance of the basin for these storms is summarized in Table 15.

The estimated construction cost of the storage/sedimentation facility was \$6,910,000 or about \$494,000/m$^3$·s (\$21,400/Mgal·d) of peak design flow capacity. The sedimentation facility was estimated to cost about \$10,600/ha (\$4,300/acre) of combined sewer area. The estimated operation and maintenance costs are about \$50,000/yr or about \$0.14/kg (\$0.06/lb) of suspended solids removed.

Table 15. PERFORMANCE OF THE SAGINAW STORAGE/SEDIMENTATION BASIN, 1978

| Storm date | Avg hydraulic overflow rate, gal/ft$^2$·d | Peak hydraulic overflow rate, gal/ft$^2$·d | Pollutant removal[a] | | | | | |
|---|---|---|---|---|---|---|---|---|
| | | | Suspended solids | | | BOD | | |
| | | | Influent, mg/L | Effluent, mg/L | Removal % | Influent, mg/L | Effluent, mg/L | Removal, % |
| 8/19/78 | 970 | 1,500 | 896 | 62 | 93 | 126 | 40 | 68 |
| 9/13/78 | 1,235 | 6,500 | 149 | 27 | 82 | 62 | 20 | 68 |
| 9/20/78 | 2,270 | 6,300 | 420 | 232 | 45 | 42 | 31 | 26 |
| Average | | | | | 73 | | | 54 |

a. From flow-weighted composite samples of influent and effluent.

Sedimentation/Biological Treatment Lagoons--

Based on projected pollutant removals of 95% for suspended solids and BOD using a series of lagoons during an EPA demonstration project in Mount Clemens, Michigan [31], a citywide combined sewage treatment system that also included storage and sedimentation was constructed around the lagoons. The treatment facilities are expected to meet effluent water quality criteria of 10 mg/L $BOD_5$ and 15 mg/L suspended solids. The projected cumulative system component process performance is shown in Figure 6.

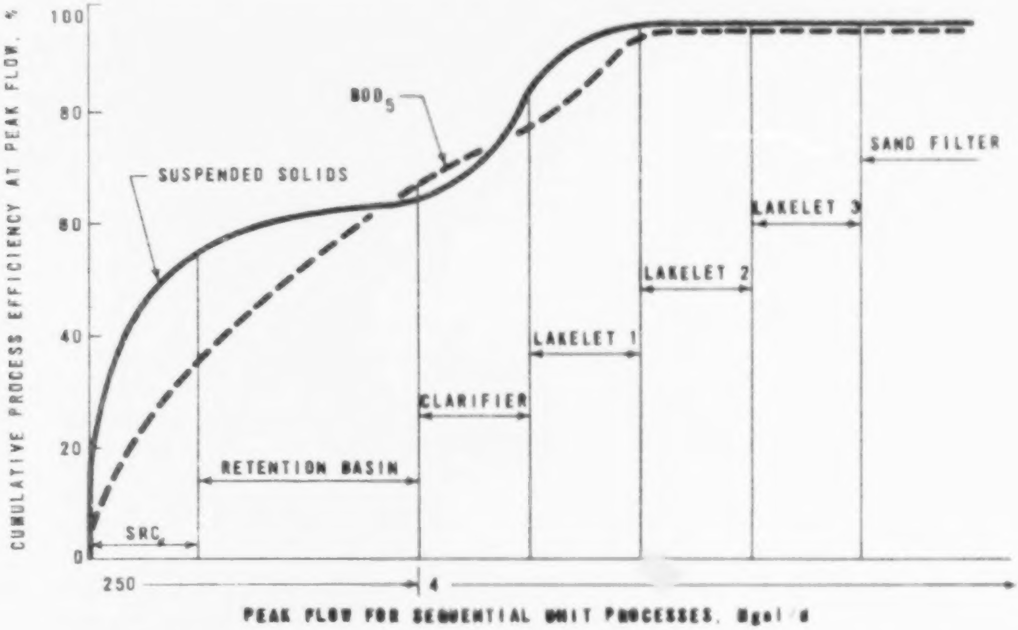


Figure 6. Projected cumulative pollutant removal efficiencies of Mount Clemens combined sewage treatment facility.

The estimated cost of the lagoon system is approximately \$1,650,000 and the cost of the sedimentation facility, including the storage basin, is \$6,930,000, for a total of about \$18,400/ha (\$7,460/acre) of combined sewer area. The estimated operational costs are about \$276,000/yr or about \$0.52/kg (\$0.24/lb) of suspended solids removed.

Swirl Concentrator/Regulator--

The swirl concentrator/regulator is designed to remove 90% or more of grit, with a specific gravity of 2.65 and an effective diameter of 0.2 mm, and settleable solids, having a specific gravity of 1.20 and an effective diameter of 1.0 mm. These solids characteristics were used in hydraulic model testing to develop design criteria for prototype installations [34]. The solids are separated by secondary fluid motion because of the long circular flow path in the swirl.

A full-scale demonstration project using a 7.3 m (24 ft) diameter swirl concentrator was constructed in Lancaster, Pennsylvania. The swirl tank design used the standard design details and relationships developed from the model testing for an inlet diameter of 0.9 m (3 ft) and a design flow of 1.13 $\text{m}^3/\text{s}$ (25.9 Mgal/d) [34]. The project also includes a swirl degritter, a grit conveyance system, instrumentation to conduct a full-scale monitoring and evaluation program, a disinfection system, and a control building. The swirl degritter is used to remove grit from the concentrated underflow of the swirl regulator/concentrator to prevent downstream deposition, protect the pumping station, and prevent grit from reaching the downstream treatment plant. The construction costs of the facilities were estimated at about \$690,000, of which the swirl costs were estimated at about \$168,000 or about \$148,000/$\text{m}^3\cdot\text{s}$ (\$6,500/Mgal·d) of design capacity. The swirl is the least expensive physical treatment process for controlling combined sewage overflow, but removals are not as good as sedimentation or dissolved air flotation. Sedimentation, for example, averages \$480,000/$\text{m}^3\cdot\text{s}$ (\$21,000/Mgal·d) and dissolved air flotation averages \$1,164,000/$\text{m}^3\cdot\text{s}$ (\$51,000/Mgal·d) of capacity [32].

Data on the performance of the Lancaster swirl operation are limited; only a few storms have been sampled [7], and fewer have flow data for mass removal determinations. The sampling data collected may not be representative of the flow at the monitoring locations because of low sampling velocities and long piping runs to the samplers.

The estimated mass removal rate for suspended solids ranged from 17 to 80% with hydraulic flow splits from 15 to 48%, respectively. The size of the storm inflow volume and the split between the concentrated underflow and the clear overflow significantly affect the overall performance of the unit. For example, a small storm could have up to 50% of its total flow volume diverted through the foul underflow, thereby netting at least a 50% mass removal independent of any concentrating effects. However, for very large storms at the same underflow rate, a relatively small fraction of the total flow volume is diverted through the foul underflow, and the corresponding mass removal by the flow split is lower.

Tests of a swirl concentrator/regulator in Syracuse, New York, indicated that the mass suspended solids reduction ranged from about 30 to 80% and averaged 52% for 11 different storms. The corresponding mass removal for the same 11 storms by a conventional regulator was estimated at about 33%. Therefore, the concentrator contributed about 20% additional removal [35].

## Integrated Systems

Areawide combined sewage overflow control requires combinations of storage and treatment technologies to provide a level of mitigation consistent with the goals of water quality improvement and optimization of cost effectiveness. The integration of these technologies may also involve existing dry-weather treatment facilities. Storage and/or treatment facilities in several large cities have been integrated with sewer separation projects.

### Storage/Treatment Integration--

Saginaw, Michigan, uses an integrated systems approach that includes inline storage, offline storage/sedimentation, and dry-weather treatment of the stored combined flows. The dry-weather plant can handle the peak wet-weather loads and flows, and the sludge dewatering system can handle the additional solids from the combined sewage overflow events.

The unit costs of this combined sewage system indicate the benefit of integrating technology. The total system unit storage costs are about 20% less than the cost of the offline storage/sedimentation basin, alone, as shown in Table 16. Similarly, without the cost-effective storage volume provided by inline storage, the size of the storage/treatment basin required for equivalent levels of control would have to be increased at the higher unit cost rate; or if the size remained the same, the effectiveness would be reduced.

Table 16. SUMMARY OF SYSTEM COSTS,
SAGINAW, MICHIGAN[a]

| Component | Storage capacity, Mgal | Area served, acres | Construction cost, $ | Storage cost, $/gal | Cost per acre served, $/acre |
|---|---|---|---|---|---|
| Storage/treatment facilities | 3.52 | 1,600 | 6,910,000 | 1.96 | 4,300 |
| Inline storage | 1.07 | 1,600 | 370,000 | 0.35 | 230 |
| Total system | 4.59 | 1,600 | 7,280,000 | 1.58 | 4,530 |

a. ENR 3000.

### Integration of Sewer Separation Programs--

Sewer separation programs have been implemented to solve specific system problems often unrelated to quality control of combined sewage overflows, yet sewer separation can be viewed as part of a total system approach. In Seattle, for example, a program of sewer separation in selected areas of the city was intended to reduce severe local flooding problems. A comparison of the cost effectiveness of the sewer separation and Seattle's inline storage system is presented in Table 17. For equivalent results of combined sewage overflow reduction, inline storage is over 18 times more cost effective than separation. However, each measure was unique to an areawide problem solution (overflow control and local flooding). The integrated system of inline storage and separation is approximately 3 times more cost effective than just separation.

Table 17. SUMMARY OF SYSTEM COSTS, SEATTLE, WASHINGTON[a]

| Control measure | Total capital costs, $ | Contributing area, acres | Annual combined overflow volume reduction, Mgal | Excess storage capacity, Mgal | Cost, $/acre | Overflow volume reduction cost, $/Mgal | Storage cost, $/gal |
|---|---|---|---|---|---|---|---|
| Inline storage | 15,753,000 | 15,000 | 600 | 22.79 | 1,050 | 26,260 | 0.69 |
| Sewer separation | 147,810,000 | 23,000 | 300[b] | -- | 6,430 | 492,700 | -- |
| Combined inline and separation projects | 163,563,000 | 38,000 | 900 | -- | 4,300 | 131,740 | -- |

a. ENR 3000.

b. Still allows stormwater volumes and pollutants to be discharged.

Mount Clemens, Michigan, also has a combined sewage control system that integrates sewer separation with interception/storage, physical treatment, and biological treatment to eliminate all untreated combined sewage overflows. Approximately half of the city was separated because of the configuration of the collection system and the potential high cost of constructing interceptors in these areas. The total cost of the system was about $26,000/ha ($10,500/acre). A comparison of the system component costs is presented in Table 18.

### Systems Approach--

Areawide control of combined sewer overflows should consider the potential cost-effectiveness benefits of a systems approach. The combination of wet-weather storage and treatment is an essential, most promising first step for effective combined sewer overflow control. The impacts of the wet-weather system on dry-weather operations is of extreme importance, particularly when the dry-weather processes are integrated with such systems as inline storage.

Table 18. SUMMARY OF SYSTEM COMPONENT COSTS, MOUNT CLEMENS, MICHIGAN[a]

| Control measure | Total capital costs, $ | Serviced area, acres | Annual combined overflow volume reduction, Mgal | Cost, $/acre | Overflow volume reduction cost, $/Mgal | Treatment cost, $/gal |
|---|---|---|---|---|---|---|
| Sewer separation | 4,019,000 | 900 | 333[b] | 4,466 | 12,070 | -- |
| Interception | 8,916,000 | 1,150 | 426 | 7,753 | 21,000 | -- |
| Treatment | -- | 1,150 | 426 | -- | -- | -- |
| Retention basin site | 6,934,000 | -- | -- | 6,030 | -- | 0.016 |
| Park treatment site | 1,653,000 | -- | -- | 1,437 | -- | 0.002 |

a. ENR 3000.

b. Still allows stormwater volumes and pollutants to be discharged.

A common requirement for a successful systems approach is the selection of storage. Inline storage should be developed initially because it is the least expensive, \$250 to \$2,500/ha (\$100 to \$1,000/acre) of combined sewered area, and is the easiest to implement. The inline storage volume in any system, however, is limited by the size and physical characteristics of the collection system. The effective use of this limited storage volume can be greatly enhanced by automatic control operation. If more storage is required than provided by the inline system, the inline system can serve as the link between upstream offline storage and downstream storage/treatment.

Offline storage should be considered as the next step to providing more storage capacity. Offline storage basins are the most common type of measure used in large cities with combined sewer overflow problems, but the costs can be up to six times greater than inline storage costs. In many situations, offline storage is combined with a sedimentation process to treat flows exceeding the storage capacity.

Another potential for storage, but which has not received great attention, is the use of source detention measures in upland areas of combined systems to prevent or delay the release of runoff into the combined sewer. The source controls that could be easily applied in a systems approach are rooftop, plaza, and parking lot storage in highly developed urban areas and source detention and detention/percolation basins in less developed areas. These measures may be as cost effective as large in-system structural controls.

The use of a dry-weather treatment facility as an element of the wet-weather control system provides additional treatment for reducing combined sewage overflow pollutants. For inline storage systems, it is the only treatment step of the system. While dry-weather facilities may help to control combined sewage (increased pollutant removal), unless they have been designed or properly evaluated for the impacts of increased combined sewage flows and loads, process operational problems could develop. This could result in the need for increased clarification capacity, sludge and grit handling facilities. The key to a cost-effective wet/dry integration may therefore depend on the dry-weather facilities capability, or lack of capability, to handle stored flows.

In a systems approach, the type, mix, and size of alternative storage/treatment technologies can provide various levels of control and overall cost effectiveness. Several methodologies are available for evaluating storage/treatment tradeoffs in planning combined sewage overflow control systems [32]. One, developed by the University of Florida, is a desktop approach for 208 level planning activities and can screen alternative stormwater mnagement plans [36]. A simplified stormwater management model is currently being documented by Metcalf & Eddy, Inc., for release by the EPA [37]. This model is a 201 facilities planning model that is an inexpensive and flexible tool for planning and preliminary sizing of stormwater facilities. The model can assess the effects of alternative storage/treatment balances developed for a systemwide control approach.

## SECTION 3

## RECOMMENDATIONS

Over the past 14 years, millions of research and development dollars have been granted by the U.S. EPA for the study of storm and combined sewer overflows. The output of this program has served as the base for implmenting large, full-scale, areawide control solutions. The emphasis of the program has principally been to control combined sewage overflows. Recent studies, however, have indicated that the potential pollution from urban runoff and other nonpoint sources has caused major pollution problems in receiving waters, ranging from nutrient enrichment, introduction of toxic materials, turbidity, and sediment deposition [1-5]. The magnitude of the urban runoff problem, therefore, may be of equal importance compared with the combined sewer overflow problem, particulary when flooding and erosion impacts are considered.

Federal research and development (R&D) programs should continue to refine and develop most promising approaches to areawide, combined sewer overflow control systems for user guidance and to advance the state-of-the-art for BMPs. Particular emphasis should be placed on program needs related to the conduct of R&D program evaluations.

### COMBINED SEWER OVERFLOW CONTROLS

Considering the large investment made in demonstration projects, and the even larger potential investment for full-scale areawide control, ranging into the billions of dollars for such systems as San Francisco's wastewater and combined sewer overflow master plan [6], and Chicago's deep tunnel and reservoir plan [1], more emphasis should be placed on obtaining information on process and system effectiveness, costs, and receiving water quality impacts/benefits from established and operating facilities. Several considerations for addressing these data needs include:

- Monitoring programs need to be continued or reestablished on existing demonstration facilities that represent most promising technology to fill data gaps and create a functional data base. Many of the demonstration projects, once built and evaluated under demonstration programs, receive little subsequent monitoring after they are turned over to the local jurisdiction.

- The data collected during the initial phases of the demonstration program evaluation period may be unrepresentative or inadequate. During the first year of operation of any facility, operational changes, system debugging, and startup problems may interfere with data collection and representative operational costs.

A number of facilities constructed under 201 programs may also be representative of most promising technology and would offer excellent opportunities for low-cost monitoring programs without the need for new costly construction of R&D facilities. Saginaw, Michigan's storage/sedimentation facility is an excellent example of a nondemonstration facility that was monitored with R&D funds.

## BEST MANAGEMENT PRACTICES

Studies of source control measures for urban runoff should be increased to provide an adequate data base for evaluating appropriate alternative control approaches. The need for such studies is pressing. A builders' association in Virginia, for example, has provided $34,000 for the study of urban runoff control practices to identify the best, least expensive methods for controlling urban runoff from new development, in actual field conditions. The key objective is to determine which BMPs are cost effective before local governments force builders to install and pay for such facilities [7]. The objectives of this private study should be followed through in R&D demonstration projects to identify and monitor the following:

- The costs and effectiveness--both flow control and pollutant removal efficiencies, design relationships, and receiving water impacts resulting from control implementation.
- The impact and benefits of multiuse facilities including those that have flood and erosion controls and recreational uses.
- A monitoring strategy to produce data that may be used and transferred for evalution of similar control methodologies.

## PROGRAM NEEDS

The foregoing recommendations all indicate the need to collect and compile reliable data allowing users/designers/decision-makers to evaluate and select cost-effective controls or control systems. The data needs are keyed to receiving water quality, control measure performance, and costs. Specific data requirements needing attention are summarized in the following tasks:

- Refine and expand systemwide cost data. This information would include both unit process or control costs and resulting systemwide unit costs where several controls are integrated to achieve the most cost-effective combinations--"knee of the curve" analysis comparing marginal costs to marginal benefits.
- Develop operation and maintenance costs from actual operating experience, not affected by startup or debugging problems. In times of high energy and labor costs, evaluation of the annual costs may contribute more to the deciding criteria for alternative selection, than just construction costs alone.

- Evaluate impacts on the dry-weather treatment plant operation, where they are integrated into the combined sewage overflow control system. Specific data requirements include effects on process performance from the increased flow and pollutant load, evaluation of required costs to modify the plant to handle wet-weather flows, and evaluation of sludge handling problems.

- Expand performance data base on the most promising control technologies. This includes evaluation of performance under transient-loading conditions, process effectiveness limits, settling characteristics and changes in water quality in storage facilities, and mass balances of pollutant removal in integrated control systems.

- Evaluate receiving water impacts/benefits using existing data (limited) and new data from monitoring programs designed to fill these data gaps. These programs should emphasize both long-term macroscopic effects and single-storm event analysis.

## SAMPLING PROCEDURES

Monitoring the effectiveness and operational parameters of a demonstration control facility is the most critical function of a facility evaluation. The collection of representative samples is essential to document the process being evaluated for future guidance. Development of a guidance program on sampling procedures would be beneficial to the evaluation of storm and combined sewer overflow demonstration facilities, and could possibly avoid some of the following problems that can limit the value and usefulness of sampled data:

- Flow velocities at the sampler intake and in the sample lines should be similar to the velocity of the flow stream being sampled. Sampling velocities of at least 1.2 m/s (4 ft/s) should allow adequate intake of heavy settleable solids and keep them in suspension; therefore, more representative samples can be taken.

- Both automatic composite and timed discrete samples should be taken on at least the influent stormwater flows for complete flow characterization. Composite samples should be adequate for the effluent from a stormwater storage or treatment unit. The discrete samples will identify first-flush or transient-loading conditions that may influence system design or operation; i.e, using or designing sufficient detention capacity to capture high initial storm loads. The automatic composite sampler should be capable of taking flow-weighted samples.

- A uniformity of monitoring analysis and procedures between different projects around the country is essential to compile and extend a useful data base that is statistically significant, such that an aggregate information source can be created and evaluated as a homogeneous unit.

- Accurate flow measurement is essential for the evaluation of system effectiveness. In addition to process efficiency, which is characterized by pollutant concentration removal, flow measurements are required to evaluate total mass removal effectiveness.
- Analysis for particle settling velocities and associated pollutants is essential to be able to select and design better treatment facilities. This information is also useful in determining solids transport characteristics.

# PART 1

# BEST MANAGEMENT PRACTICES

## SECTION 4

## PUBLIC UTILITY APPROACH TO URBAN RUNOFF CONTROL BELLEVUE, WASHINGTON

Establishing a public utility to control urban runoff in Bellevue, Washington, has provided flexibility in developing and financing a water quality improvement program to meet the needs of the community. In addition to enforcing stormwater regulations and control requirements on all new development, the utility is responsible for maintaining the city's drainage system. This includes making improvements that affect the water quality and the use of the natural streams as a water resource.

Financing of the utility is based on a service charge on all developed and undeveloped property. The amount of the service charge depends on the property's contribution to the runoff problem. The utility has been successful in establishing a high level of commitment to local stormwater problems and in maintaining and protecting the nature and quality of the natural stream/drainage system environment.

Both controls for erosion and sedimentation from construction sites and postdevelopment runoff management are required for the rapidly developing urban area. Major drainage system improvements, including offline and instream storage/detention, channel lining and cleaning, and stormwater drains and bypasses, are part of a comprehensive drainage master plan. The estimated costs of these master plan improvements average about $2,500/ha ($1,000/acre). The comprehensive drainage master plan is just being implemented, but has increased public awareness of water quality problems and goals, and has provided environmental and socioeconomic benefits.

An ongoing two year BMP evaluation in Bellevue is being jointly sponsored by the U.S. EPA's Storm and Combined Sewer Section and the 208 Planning Section, and the U.S. Geological Survey (USGS). The USGS will be primarily responsible for data collection to evaluate storm runoff flow and characteristics (wet-weather washoff and modeling). The EPA project will evaluate BMPs on a macroscopic scale involving analysis of 40 to 90 storms to determine basinwide effectiveness and long-term water quality impacts.

### APPROACH TO RUNOFF CONTROL

Rapidly developing urban areas can create water quality and quantity problems, and while controls on new development at least hold the problems in check, regional or areawide controls may be required to improve water quality in existing developed areas. Bellevue's Storm and Surface Water Utility is a unique, separately funded organization that regulates controls on new developments and provides a citywide stormwater control approach.

Area Characteristics

Bellevue is a rapidly developing suburban community located about 9.7 km (6 mi) east of Seattle, between Lake Washington and Lake Sammamish, as shown in Figure 7. The city encompasses about 7,700 ha (19,000 acres), of which nearly 70% is developed [1]. Dynamic growth has occurred in Bellevue over the past 25 years with almost a 16-fold increase in population: from about 5,000 in 1954 to 80,000 in 1979.

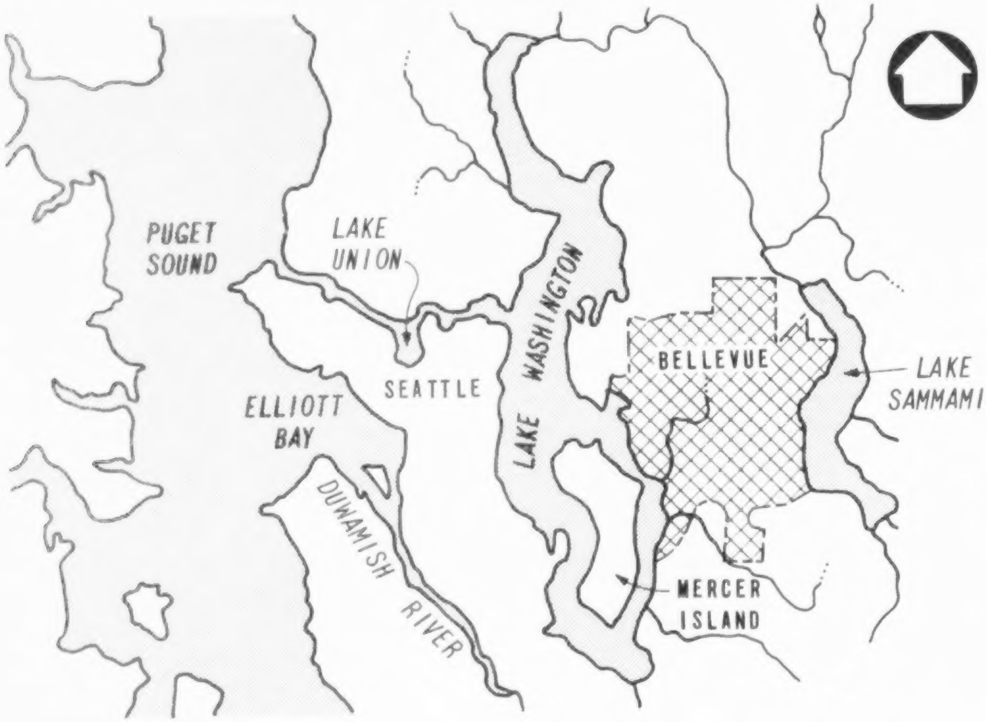


Figure 7. Location of Bellevue, Washington.

The topography of the area is rolling hills and valleys with elevations varying from approximately 7.6 to 122 m (25 to 400 ft). The area is drained by an extensive stream system that serpentines through the city. The 11 drainage basins in the city are generally small, averaging less than 610 ha (1,500 acres), and most residential developments are near small streams or creeks.

The soils are generally gravelly loam with high permeability, but there are some peat-filled valleys in the lower drainage areas and wetlands. Peat depths have been measured up to 30 m (100 ft).

The climate of the area is moderated by the Pacific Ocean and is considered maritime. The annual precipitation is about 102 cm (40 in.), of which 77% occurs between October and March [2].

The land use in Bellevue is almost 55% single-family residential. During the past 5 years, high density residential and commercial development has increased, but single-family home construction has continued at a high level as well. The 1974 land use and the projected future land use changes are summarized in Table 19.

Table 19. 1974 LAND USE AND PROJECTED FUTURE LAND USE, BELLEVUE [1].

| Land use | Estimated 1974 land use, acres | Projected future land use, acres | Change in land area, acres | Change in total land area, % |
|---|---|---|---|---|
| Single-family residential | 9,743 | 10,280 | 537 | 2.7 |
| Multifamily residential | 517 | 1,173 | 656 | 3.4 |
| Commercial | 1,116 | 2,862 | 1,746 | 9.1 |
| Light industrial | 463 | 772 | 309 | 1.6 |
| Institutional | 583 | 713 | 130 | 0.6 |
| Parks, open space, undeveloped | 6,175 | 2,680 | -3,495 | -18 |
| Freeways/roads | 663 | 780 | 117 | 0.6 |
| Total | 19,260 | 19,260 | 0 | 0 |

## Problem Assessment

The rapid development in Bellevue has created stormwater runoff problems in most of the natural streams draining the area, including:

- Flooding
- Erosion
- Stream sedimentation/siltation
- Water quality problems

Flooding, erosion, and sedimentation are a direct result of increased runoff, extreme topographic relief (going from steep ridges to flat lowlands), high groundwater table in the lowlands, and poor soil conditions. The water quality problems created by the increased runoff include the effects of pollutants entering the streams and receiving waters, and the changes in the ecology or nature of the stream. Bellevue's principal problem areas are identified in Figure 8.

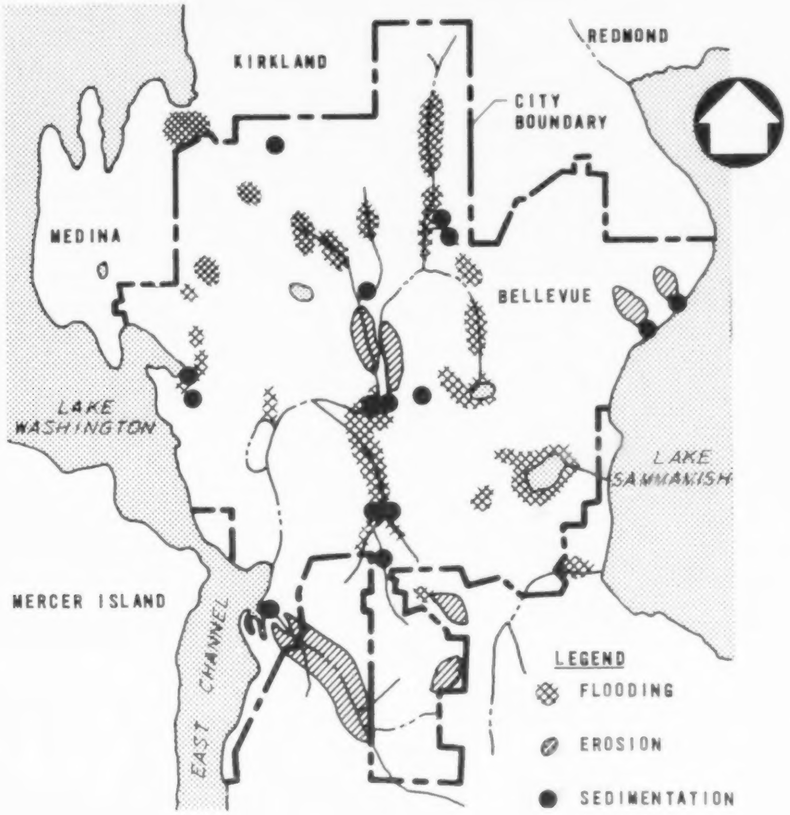


Figure 8. Urban runoff problem areas, Bellevue [1].

Quantity--

Most of the streams in Bellevue are still above ground and exposed to development, although some channelization and underground pipes have been installed. Flows in these streams are generally less than 0.08 m$^3$/s (3 ft$^3$/s); however, during intense rainfall, flood flows have exceeded 11 m$^3$/s (400 ft$^3$/s). Estimates of the increases in runoff rates above predevelopment rates for several different land uses are summarized in Table 20.

Table 20. ESTIMATED INCREASES IN RUNOFF FROM DEVELOPMENT OVER PREDEVELOPMENT RATES [1]

| Land use | Runoff increase, % |
|---|---|
| Single-family residential | 200 |
| Multifamily residential | 800 |
| Commercial | 1,900 |
| Industrial | 1,900 |

The flooding caused by these increased flows is the most visible problem and occurs along stream channels and in low lying wetlands. Streets and basements have also been flooded.

Erosion from cleared land under development and from stream channels has also occurred. Besides loss of land and soil and damage to stream channel banks and vegetation, sedimentation and siltation of the stream bottom can change the biological structure of the benthic life in the stream and reduce the flow capacity of a stream segment, possibly causing additional flooding. Deposits of eroded sediment can cover fish spawning beds and kill the aquatic insect community that supports the fish.

### Quality--

Although water quality in Lake Washington has improved since the late 1950s and early 1960s, mainly because of the diversion and regional treatment of sanitary sewage, nutrient discharges from small streams can cause nearshore water quality problems. The estimated nutrient loadings from Kelsey Creek, the major stream draining Bellevue, are summarized in Table 21. The increase in load is principally a result of increased urban development. Other nearshore problems include increased sedimentation, turbidity, and toxic inputs of oils, heavy metals, and pesticides. In addition to these urban runoff pollutants, ambient stream dissolved oxygen, temperature, and turbidity are prime water quality considerations because preserving the chemical and physical characteristics of the streams and wetlands for fish propagation is one of Bellevue's goals.

Table 21. ESTIMATED AND PROJECTED
NUTRIENT LOADS IN KELSEY CREEK [3]

| Nutrient | 1970 load, lb/d | 2000 projected load, lb/d | Increase, % |
|---|---|---|---|
| Phosphorus as $PO_4$ | 8.2 | 15.8 | 93 |
| Nitrogen as $NO_3$ | 58.7 | 142.0 | 142 |

## Approach

Bellevue uses low and nonstructural source controls to control urban runoff. Regulation, enforcement, and implementation of these controls is conducted by a utility of the city government. The utility receives public financing through the collection of service charges and is a major division of the Public Works Department.

### Utility Concept--

A sequence of events, beginning in the early 1970s over concern for water quality in the streams draining the city, led to the adoption of an ordinance regulating clearing and grading operations to minimize sediment and a recommendation that a Storm and Surface Water Utility be established to assume responsibility for all water resource related matters. The utility was established in 1974 and is a most promising approach for runoff control on a local level. One of the significant problems of the utility was developing an equitable billing system and rate structure [4].

Utility Organization and Function--The responsibility of the utility is not only to control the drainage system, but to protect the water resource itself, as the drainage system affects it. The utility focuses on a number of diverse issues, such as biological quality, aesthetics, and recreational benefits as well as conveyance of stormwater runoff. To achieve these goals, the utility was originally organized to provide the required technical and operational staff in a single integrated group.

This organizational structure is significantly different from that found in most public works departments and is often difficult to form within established agencies, particularly those without funds earmarked for drainage. Most public works departments are organized in a staff concept, with an engineering division providing all engineering services to other divisions, a maintenance division providing all maintenance services to the department, and a construction division providing inspection services, and so on. A staff concept can be unresponsive to the needs of a stormwater management program because stormwater control is only one of several responsibilities assigned to a division, and usually carries a secondary priority to road construction, water and sewer installation, and construction inspection services.

Bellevue's Storm and Surface Water Utility was organized to provide utility inspectors, plan review engineers, water quality technicians, and maintenance personnel under a single operation for controlling urban runoff and water resource problems. This approach has been very successful in Bellevue and has proved an efficient mechanism for enforcing and financing urban stormwater management programs.

Utility Financing--Financing stormwater management programs at the local or city level has often been difficult, and limited funding has restricted the effectiveness of program control and enforcement. In Bellevue, it was recommended that storm and surface water activities be treated as a utility and financed similar to other utility operations.

Storm and surface water services are, however, different from most other utility services, such as water supply or sewage utilities where the public receives a readily identifiable product or service. One of the biggest problems faced in establishing the utility was convincing the public that runoff from developed land entering a drainage system created problems and that the service performed by the utility, although not readily visible, could benefit the entire public and should therefore be subsidized by all property owners.

The utility rate structure in Bellevue is based on a property's contribution to the stormwater problem with the level of charge commensurate with (1) the property area, and (2) the intensity of development. A rate structure determined solely on the property's contribution to the problem, however, does not provide for such situations as oversizing downstream drainage controls in expectation of future upstream development. Owners of upstream undeveloped property could argue that they were not contributing to the problem, which would force downstream owners to pay for the overdesign.

Another important element of the rate structure is that the utility charges apply to both public and private property. Even streets and freeways are billed as developed, real property.

The utility is now well established and accepted by the community. The residential bimonthly service charges for the utility average about $1.60. About $600,000 is generated annually to carry out the urban runoff and water resources programs and represents a stable source of funding.

Facility Design Criteria--

Stormwater control facilities are designed to limit the rate of runoff from developed areas to predevelopment rates and store runoff in excess of this rate. Infiltration potential and impervious surface characteristics are also considered in the design criteria [5].

Allowable Runoff Rate--The allowable runoff rate is based on a citywide average of predevelopment runoff from a 10 year storm of 0.014 $m^3$/ha·s (0.2 $ft^3$/acre·s), or about 0.5 cm/h (0.2 in./h). The runoff criteria also apply to the release rate from storage facilities.

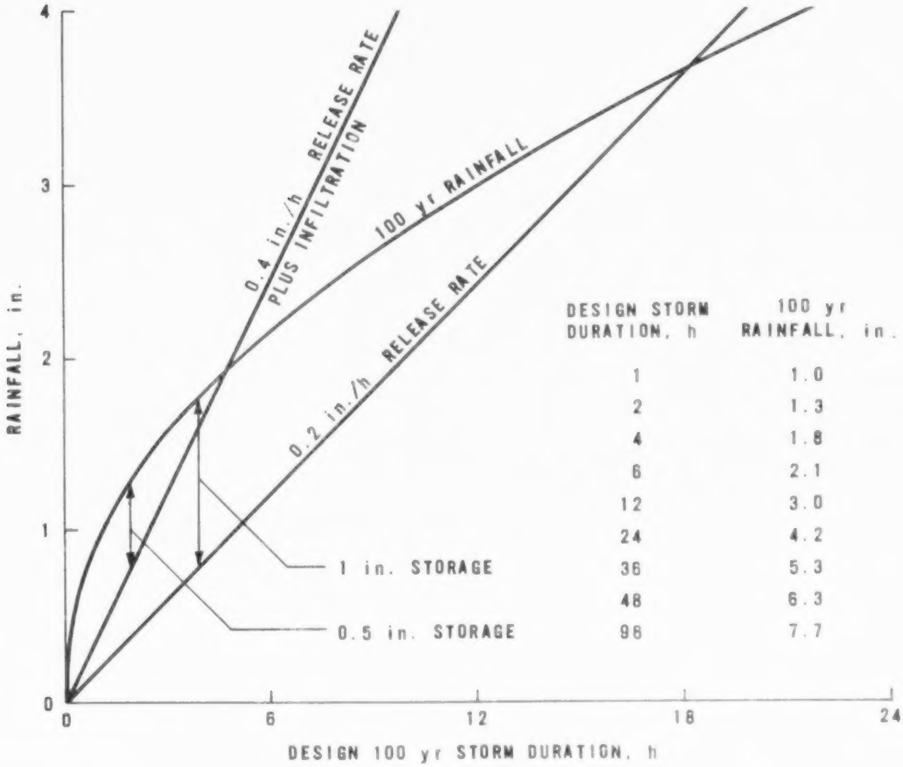
Storage Requirements--Storage requirements for runoff in excess of the allowable runoff rate are based on a rainfall intensity/duration curve developed for the 100 yr storm, shown in Figure 9. Impervious and pervious areas have different storage requirements.

For impervious areas, the storage required is based on a 100 yr, 4 h rainfall of 4.6 cm (1.8 in.) less the allowable release rate over the 4 h period, or a total storage requirement of 2.54 cm (1.0 in.).

For pervious areas, the storage required is based on a 100 yr, 2 h rainfall of 3.3 cm (1.3 in.) less the allowable release rate over the rainfall period and the infiltration rate over the rainfall period. Storage would not be required for pervious areas with infiltration rates in excess of 1.3 cm/h (0.5 in./h). However, because of the poor infiltration conditions in most areas of the city, storage may be required in many situations. In the absence of infiltration test data for a particular development site, an assumed allowable infiltration rate of 0.5 cm/h (0.2 in./h) is used. At this rate, the total storage requirements for pervious areas is about 1.3 cm (0.5 in.). Different infiltration rates would affect the total storage requirement.

## RUNOFF CONTROL FACILITIES

Runoff control facilities are required both during construction for erosion and sediment control and after development for runoff control. A large emphasis is placed on the use of nonstructural controls; however, in many situations the use of low structural controls, such as storage basins, is required.



| DESIGN STORM DURATION, h | 100 yr RAINFALL, in. |
|---|---|
| 1 | 1.0 |
| 2 | 1.3 |
| 4 | 1.8 |
| 6 | 2.1 |
| 12 | 3.0 |
| 24 | 4.2 |
| 36 | 5.3 |
| 48 | 6.3 |
| 96 | 7.7 |

Figure 9. Storage volume design curve for the 100 yr storm, Bellevue [5].

## Sediment Controls

Sediment controls are required in all new construction areas, where the soil has been disturbed or vegetation removed to prevent sediment from reaching the receiving streams. Simple sediment trap ponds are the most common and are usually removed after construction is completed. These ponds are located downstream from the construction site and runoff is directed to the pond along protective berms or through drainage ditches. The required pond capacity is at least 0.028 $m^3$ (1 $ft^3$) for each 4.6 $m^2$ (50 $ft^2$) of tributary area. Temporary sedimentation ponds are shown in Figure 10.

## Permanent Runoff Controls

Permanent runoff controls use some form of detention with offline dry ponds and parking lot storage being the most common. Other detention methods used include rooftop storage (limited use), instream storage, underground vaults, and underground pipes. Infiltration and recharge trenches can be used to reduce the storage requirements.

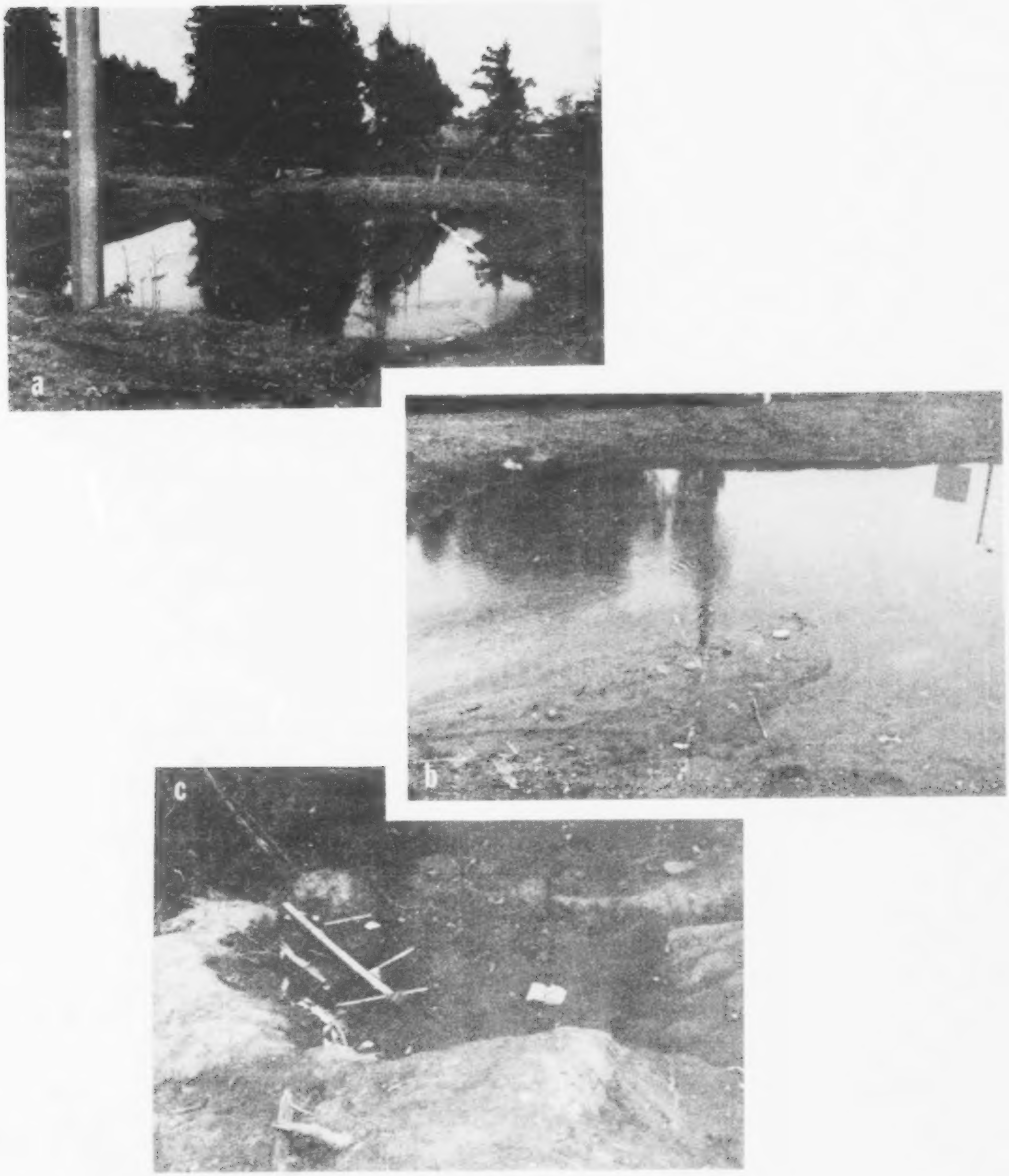


Figure 10. Temporary sedimentation ponds for construction sites: (a) runoff from large construction site channeled to pond, (b) sediment trapped in pond, and (c) small sediment pond for apartment complex under construction.

A number of small offline detention ponds have been used for residential and commercial applications. Most are dry ponds and are simple excavations, as shown in Figure 11. They can have protected inlets and inverted outlets to prevent trash from entering the pond and to prevent floatables and oil and grease from leaving the structure.

In addition to detaining stormwater, they also act as sediment traps and require cleaning and maintenance as the sediment deposits increase.

An example of an offline detention system used to control runoff volumes from about 6.1 ha (15 acres) of almost totally impervious street surface is shown in Figures 12 and 13. A series of five detention basins, separated by control manholes, was constructed as part of a four-lane avenue reconstruction project and was integrated into a roadside park setting.

Runoff from approximately 1,070 m (3,500 ft) of roadway is drained into the pond system through a 68 cm (27 in.) pipe that interconnects the control manholes around each pond. A base flow runoff is allowed to pass through the pond system. However, as runoff increases, the control manholes impede the flow and sequentially fill the ponds through an inlet/outlet grate, beginning with the upstream pond. Weirs in the manholes prevent the storage depth from overtopping the pond embankment and pass the flow to the next downstream pond.

The pond system can be adjusted for various flow and operating conditions. The base flow passes through the manholes through adjustable mud valves in the floor to the next downstream manholes. The weirs controlling the maximum pond elevation are also adjustable. The ponds can also be operated as a storage/infiltration system using a 15 cm (6 in.) underdrain pipe connected to the downstream side of each manhole. The underdrain system can provide a high level of treatment in addition to volume and rate control.

Instream detention has also been used. Existing or excess capacity in the stream itself or instream ponds provide additional storage volume, as shown in Figure 14. Check dams with adjustable stop log weirs regulate the available storage and also provide some stream aeration from the falling water. Floating booms across stream channels have also been used to trap floatable material.

## COSTS

While a large number of stormwater controls will be constructed and financed by private developers, the City of Bellevue has developed a drainage master plan that identified needed improvements to the drainage system over the city and also recommended a number of regional detention facilities to control runoff volumes and rates from several large drainage areas [1]. The estimated total cost for the master plan improvements is about $21 million, of which Bellevue's cost is about $19.6 million (ENR 3000). The difference in cost is to be shared by surrounding county and city agencies. Through the areawide 208 agency, Bellevue is developing intergovernmental agreements for drainage control where the drainage basins cross jurisdictional lines.

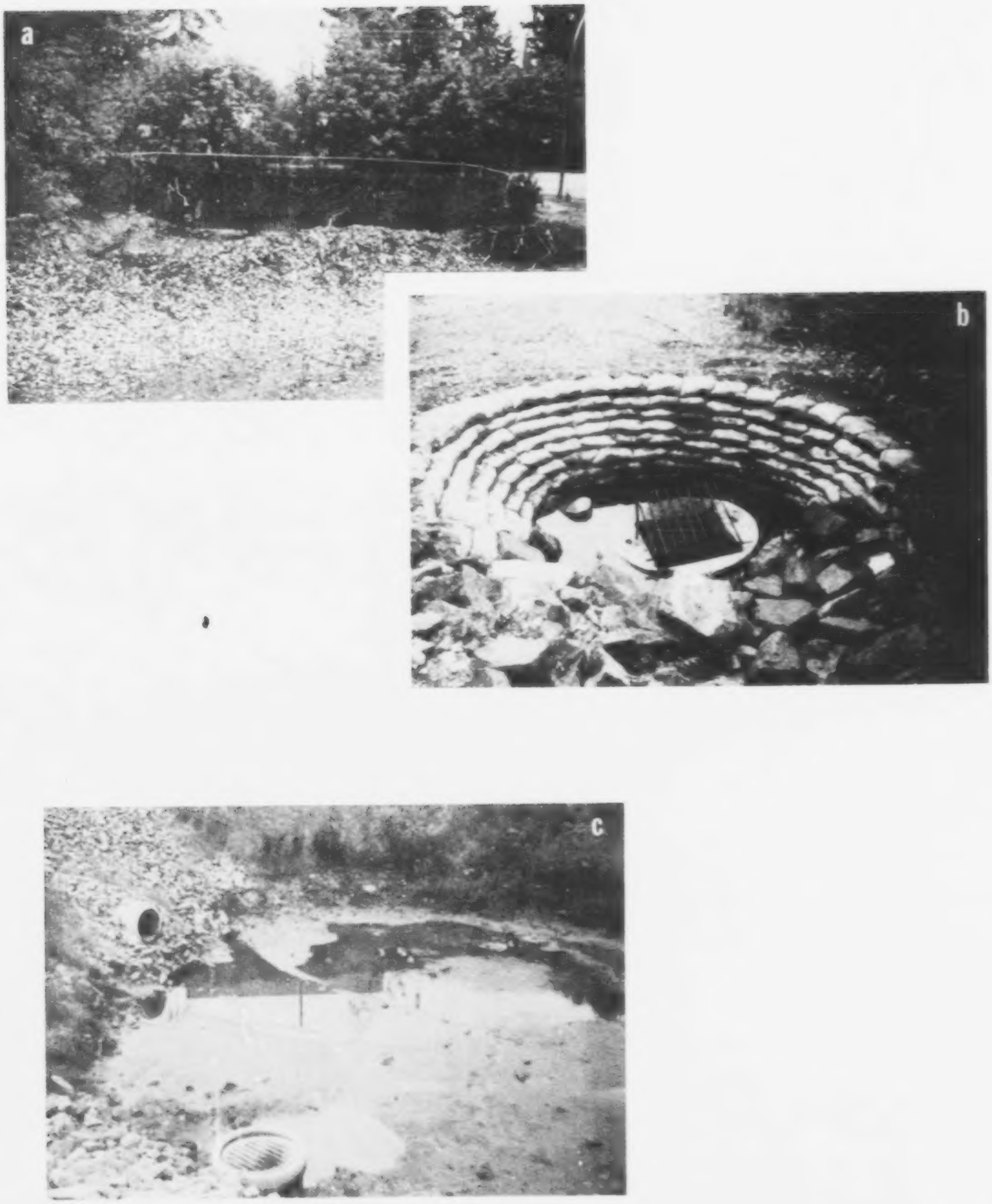


Figure 11. Permanent stormwater detention ponds: (a) detention pond with fence in developing residential area, (b) bar rack inlet to detention pond to trap debris, and (c) oil-covered detention pond receiving runoff from a bus garage and storage area.

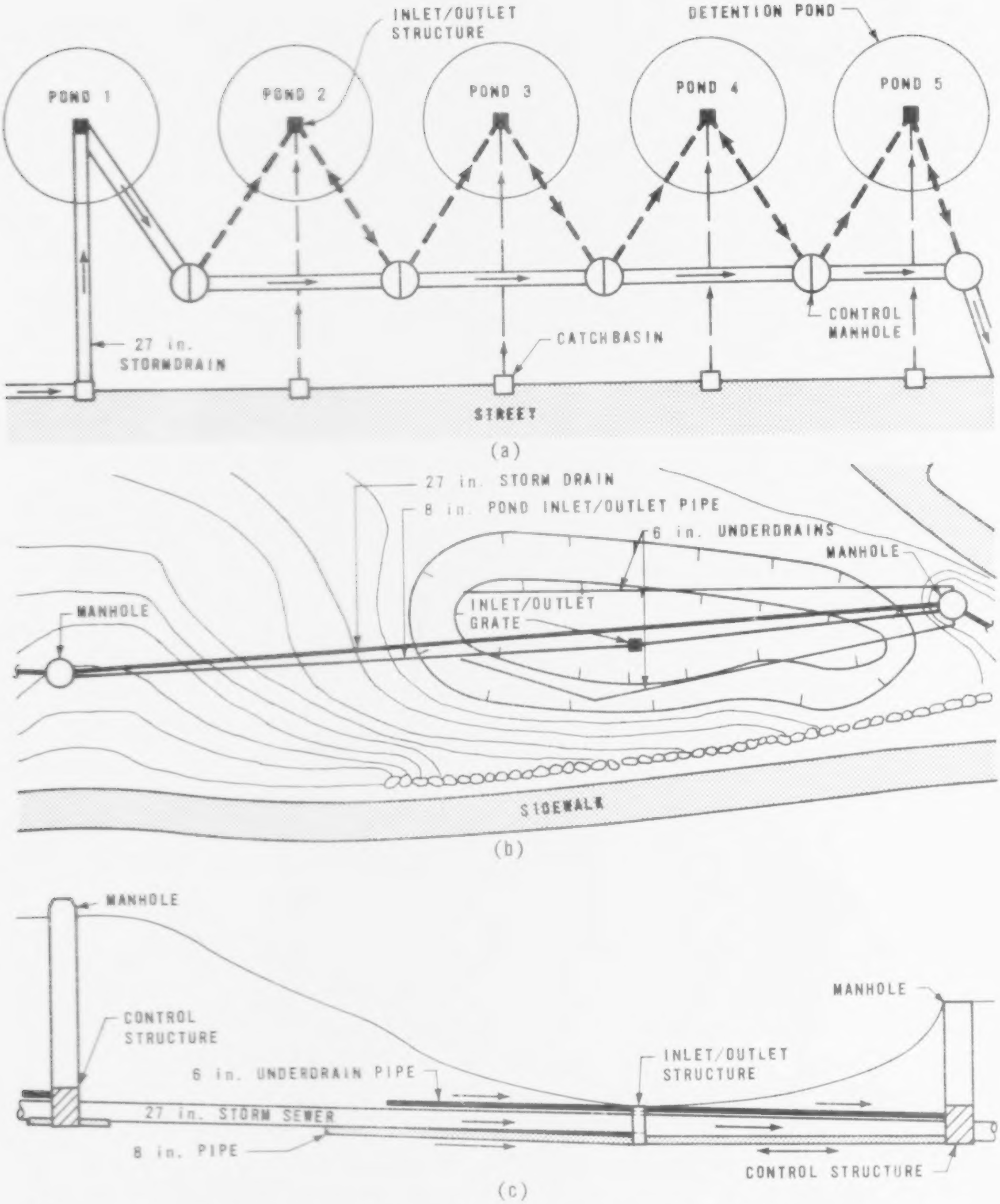


Figure 12. Offline detention system to control runoff from about 15 acres of road surface: (a) schematic plan showing five offline detention ponds in series; (b) detail of Pond 5 showing stormdrain, control manholes, inlet/outlet pipe, and underdrain; and (c) section view of Pond 5.

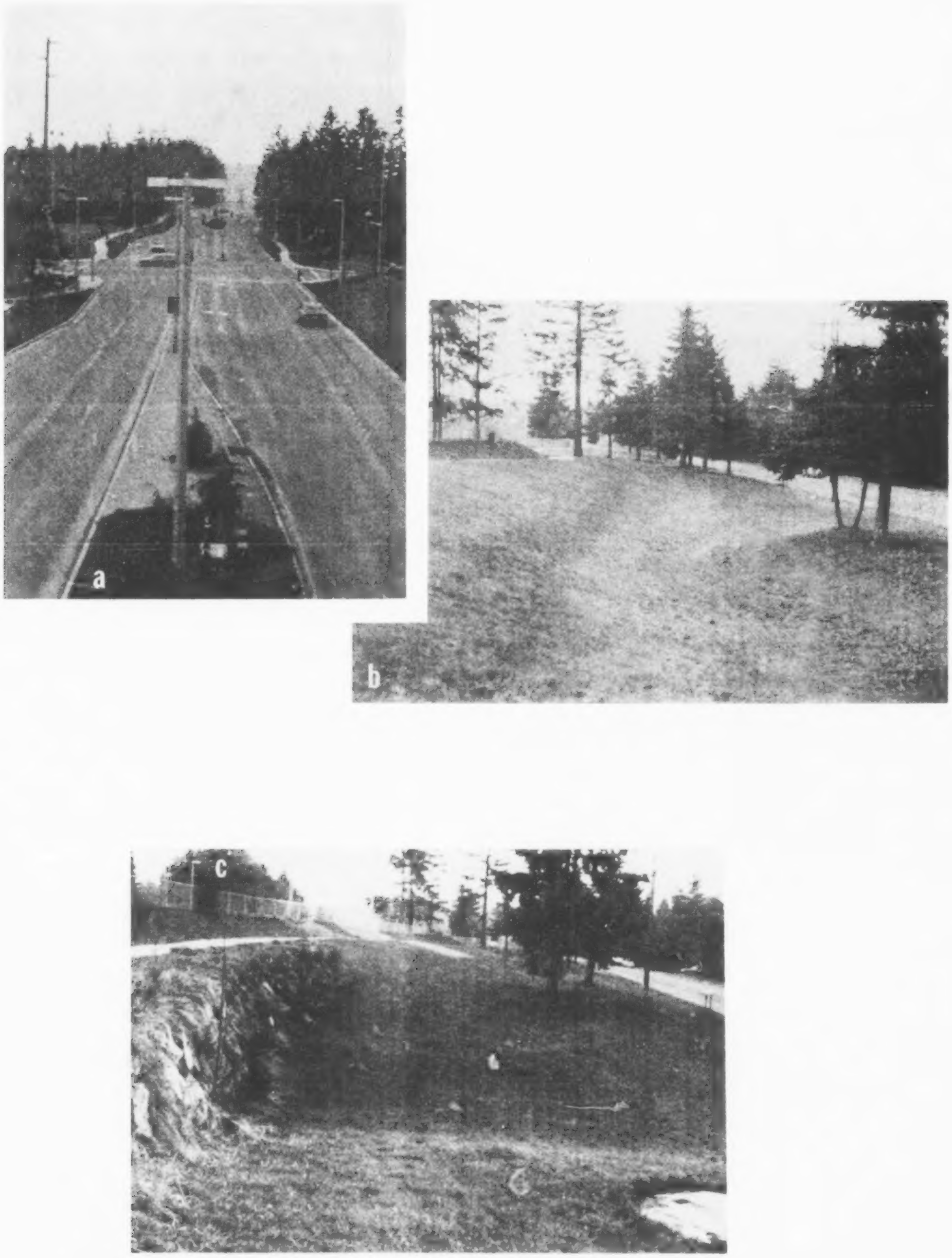


Figure 13. Offline detention system: (a) drainage area of impervious roadway, (b) detention ponds consist of grassed depressions that form a roadside park, (c) detention Pond 5 with control manhole in right foreground.

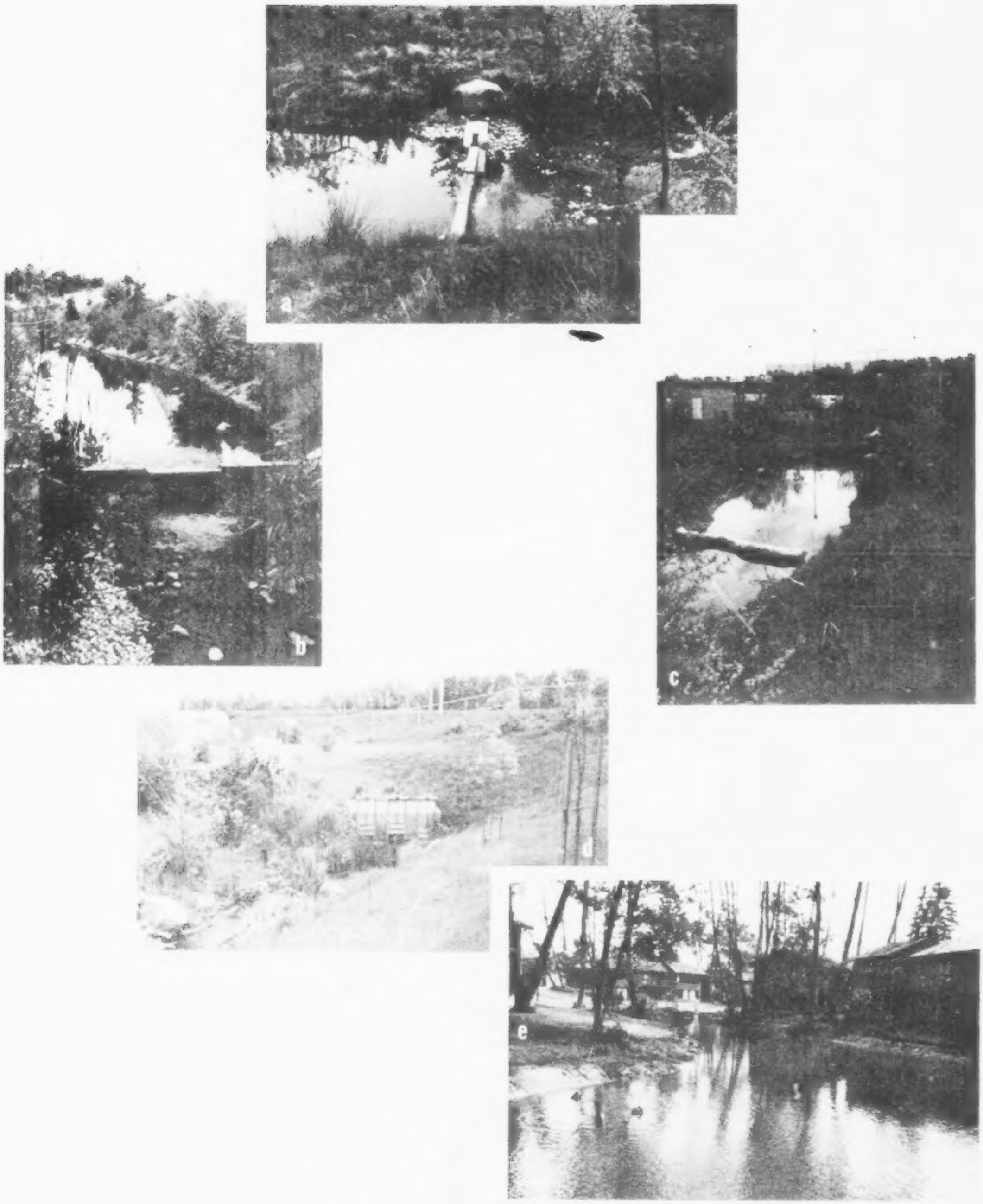


Figure 14. Instream stormwater storage: (a and b) check dams in relocated streambed, (c) floating boom to trap floatables, (d) adjustable sluice gates in stream, and (e) instream permanent pool storage pond controlled by check dams in a residential area.

Storage pond construction and drainage system improvements are planned over a phased program:

- Phase I - immediate action. This phase focuses primarily on the purchase of land for regional detention facilities. Land purchase and/or construction improvements, which will take 4 years, are planned for seven of the eight major drainage basins in Bellevue at a cost of $8.4 million.

- Subsequent phases. The next level of land purchase and construction improvements will be prioritized during Phase I for subsequent phases as yet unspecified.

The drainage system improvements include channel modification, streambank protection, channel cleaning, installation of parallel pipes and bypass pipes, and culvert replacement.

## Storage Pond Costs

Two types of storage ponds are planned: (1) excess storage capacity in streams and wetlands is used; only minor construction of embankments or modification to culverts is required; and (2) storage capacity is created by extensive excavation and construction of embankments. Construction cost estimates of the 19 storage ponds planned in the Kelsey Creek system are presented in Table 22.

Table 22. ESTIMATED STORAGE POND CONSTRUCTION COSTS, KELSEY CREEK DRAINAGE SYSTEM [1][a]

| Kelsey Creek subbasin | Ultimate storage capacity, $ft^3$ | Construction cost, $[b] | Unit storage cost, $/$ft^3$ |
|---|---|---|---|
| Kelsey Creek - 4 pond sites | | | |
| 1 | 348,000 | 78,000 | 0.22 |
| 2 | 1,177,000 | 62,000 | 0.05 |
| 3 | 3,049,000 | 739,000 | 0.24 |
| 4 | 3,267,000 | 34,000 | 0.01 |
| Valley Creek - 4 pond sites | | | |
| 1 | 936,000 | 121,000 | 0.13 |
| 2 | 2,483,000 | 319,000 | 0.13 |
| 3 | 2,614,000 | 539,000 | 0.21 |
| 4 | 1,350,000 | 694,000 | 0.51 |
| West Tributary - 5 pond sites | | | |
| 1 | 1,111,000 | 448,000 | 0.40 |
| 2 | 283,000 | 184,000 | 0.65 |
| 3 | 1,786,000 | 22,000 | 0.01 |
| 4 | 1,459,000 | 30,000 | 0.02 |
| 5 | 327,000 | 11,000 | 0.03 |
| Richards Creek - 4 pond sites | | | |
| 1 | 2,309,000 | 22,000 | 0.01 |
| 2 | 828,000 | 19,000 | 0.02 |
| 3 | 2,831,000 | 506,000 | 0.18 |
| 4 | 1,699,000 | 15,000 | 0.01 |
| Mercer Slough - 2 pond sites | | | |
| 1 | 675,000 | 15,000 | 0.02 |
| 2 | 871,000 | 91,000 | 0.10 |

a. ENR 3000.

b. Includes contingency, design, overhead, and administration.

The average cost of wetlands storage, \$1.06/m$^3$ (\$0.03/ft$^3$), is about 10% of the cost of storage ponds requiring substantial excavation, about \$10.60/m$^3$ (\$0.30/ft$^3$). Using these estimates and estimates of planned storage ponds in other drainage basins, generalized unit construction cost curves were developed and are shown in Figure 15.

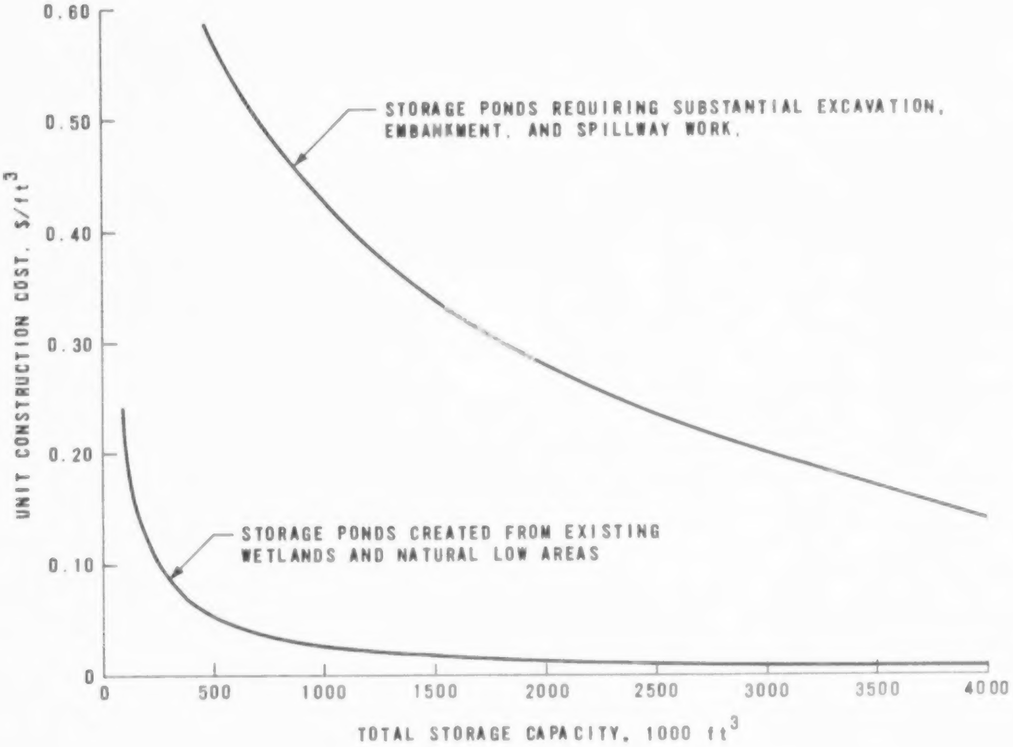


Figure 15. Storage pond construction costs, ENR 3000.

Approximately 86 ha (212 acres) of land would have to be obtained for construction of the ponds on the Kelsey Creek system, at an estimated cost of about \$4.6 million, or about \$54,000/ha (\$22,000/acre). This includes associated administration and acquisition costs [1].

Operation and Maintenance Costs

The annual operating budget of the Storm and Surface Water Utility is approximately \$600,000, which covers utility expenses and operation and maintenance of the drainage system. This revenue, based on the utility rate structure, is balanced to just meet the costs of the utility.

The rate structure charges property owners based on total area and degree of development. There are five categories of development [6]:

- Undeveloped - Real property undeveloped and unaltered by buildings, roads, impervious surfaces, or other physical improvements that change the hydrology of the property from its natural state.
- Light Development - Developed real property that has impervious surfaces of less than 20% of the total property area.
- Moderate Development - Developed real property that has impervious surfaces between 20 and 40% of the total property area.
- Heavy Development - Developed real property that has impervious surfaces between 40 and 70% of the total square property area.
- Very Heavy Development - Developed real property that has impervious surfaces of more than 70% of the total property area.

A portion of the service charge rate structure for several categories of development and property size is shown in Table 23.

Table 23. PORTION OF THE ANNUAL STORM AND SURFACE WATER UTILITY BILLING STRUCTURE[a] [6]

| Property size, acres | Billing rate by development classification, $ | | | | |
|---|---|---|---|---|---|
| | Undeveloped | Light | Moderate | Heavy | Very heavy |
| 0.25 | 5.64 | 8.4 | 11.28 | 16.80 | 22.44 |
| 0.50 | 10.32 | 15.48 | 20.64 | 30.84 | 41.16 |
| 0.75 | 15.96 | 23.88 | 31.92 | 47.76 | 63.60 |
| 1.00 | 20.64 | 30.84 | 41.28 | 61.80 | 82.32 |
| 2.00 | 41.16 | 61.80 | 82.44 | 123.60 | 164.76 |

a. Based on 1977 rates.

## IMPACTS

Bellevue's stormwater runoff control program is in its early stages of development, and while new development is required to control runoff, the master plan drainage improvements have not been completed. There has been, however, increasing public concern over the water quality and the stream environment. Local property owners can now identify the measures implemented to control urban runoff and have recognized the environmental and socioeconomic impacts of the drainage plan and the public utility.

### Environmental Impacts

The main goals of stormwater control are to protect the stream and wetlands environment and reduce potential flooding and damage caused by increased runoff from developed areas.

During development of the master plan stormwater storage and drainage system improvements, alternative plans were evaluated according to a number of environmental factors. These included (1) potential for environmental damage during high flow conditions; (2) alteration of or the need to change the natural drainage capability of streams, lakes, and wetlands; (3) potential impacts on the groundwater system; (4) surface water quality impacts including temperature, dissolved oxygen, and turbidity; and (5) evaluation of the impacts on wildlife, aquatic life, and vegetation.

With the application of low and nonstructural runoff control technology, the appearance and nature of the streams in Bellevue have improved, or at least not worsened because of the rapid increase in new development. While the effectiveness of these controls has not been monitored, the water quality in the large receiving water lakes surrounding Bellevue indicates that water quality has stabilized.

Much of the water quality improvement in these lakes was a result of the diversion of all sanitary wastewater flows. However, after sanitary flows were diverted, the emphasis and potential quality impacts of urban runoff increased, as shown in Table 24 for phosphorus in Lake Sammamish. Further water quality improvement would therefore necessarily be centered on control of urban runoff.

Table 24. PHOSPHORUS LOADS TO LAKE SAMMAMISH BEFORE AND AFTER DIVERSION OF SANITARY FLOWS [7]

|  | Phosphorus load, lb/yr |  | Percent of total |  |
|---|---|---|---|---|
| Source | Before diversion | After diversion | Before diversion | After diversion |
| Waste discharges | 16,500 | 1,100 | 37 | 4 |
| Runoff | 25,300 | 25,300 | 58 | 88 |
| Precipitation | 2,200 | 2,200 | 5 | 8 |
| Total | 44,000 | 28,600 | 100 | 100 |

Similar trends can also be seen in Lake Washington. The phosphorus load in the lake sharply decreased after sewage diversion in 1968, as shown in Table 25. Most of the phosphorus load after 1970 is a result of stormwater discharges and stream inflows.

### Socioeconomic Impacts

The socioeconomic impacts of the required runoff controls and the development of the stormwater utility have been mixed; but through a public education program, the stormwater program is well accepted and supported. The principal socioeconomic impacts are aesthetics and multiuse potential of the controls. Most of the major drainage system improvements are planned as instream or wetlands controls; therefore, there will be no displacement of people.

Table 25. ESTIMATES OF ANNUAL PHOSPHORUS LOADS TO LAKE WASHINGTON[a] [8]

| Year | Lake inflow, million $ft^3$ | Total phosphorus load, $lb/ft^2 \cdot yr$[b] | Dissolved phosphorus load, $lb/ft^2 \cdot yr$[b] |
|---|---|---|---|
| 1957 | 34.4 | 0.25 | 0.22 |
| 1962 | 34.1 | 0.53 | -- |
| 1964 | 54.9 | 0.47 | -- |
| 1970 | 42.7 | 0.10 | -- |
| 1971 | 54.4 | 0.09 | -- |
| 1972 | 53.5 | 0.20 | 0.05 |
| 1973 | 31.7 | 0.06 | 0.03 |
| 1974 | 47.0 | 0.10 | 0.04 |
| 1975 | 52.3 | 0.15 | 0.02 |

a. Does not include Seattle's storm sewer or combined sewer discharges.

b. Based on a lake area of 943,100 $ft^2$.

Maintaining the natural appearance of the open streams is a key element in the control and improvement of the drainage system. Controls on private land and new developments are reviewed for consistency of scale and materials used in relationship to the surrounding land use, topography, and general aesthetics.

The city and a number of developers have incorporated stormwater controls into multiuse facilities. These include permanent pool, instream detention ponds at a condominium development and a series of dry, offline detention ponds integrated into a recreational park along a major avenue, as shown in Figure 16.

The development of the utility provided a public educational benefit that has also aided in the control of stormwater pollutant loads to receiving waters. During the creation of the utility, a public vote on alternative financing methods indicated that the public was disenchanted about receiving a utility bill for stormwater management services. Subsequent voter approval of the utility rate structure occurred after a public education program explained the financing alternatives and the expected water quality benefits of controlling runoff. Because of the increased public awareness of potential costs and water quality, there has been significantly less dumping of oil and debris in catchbasins and neighborhood housekeeping practices, such as cleaning up pet wastes and litter control, have increased.

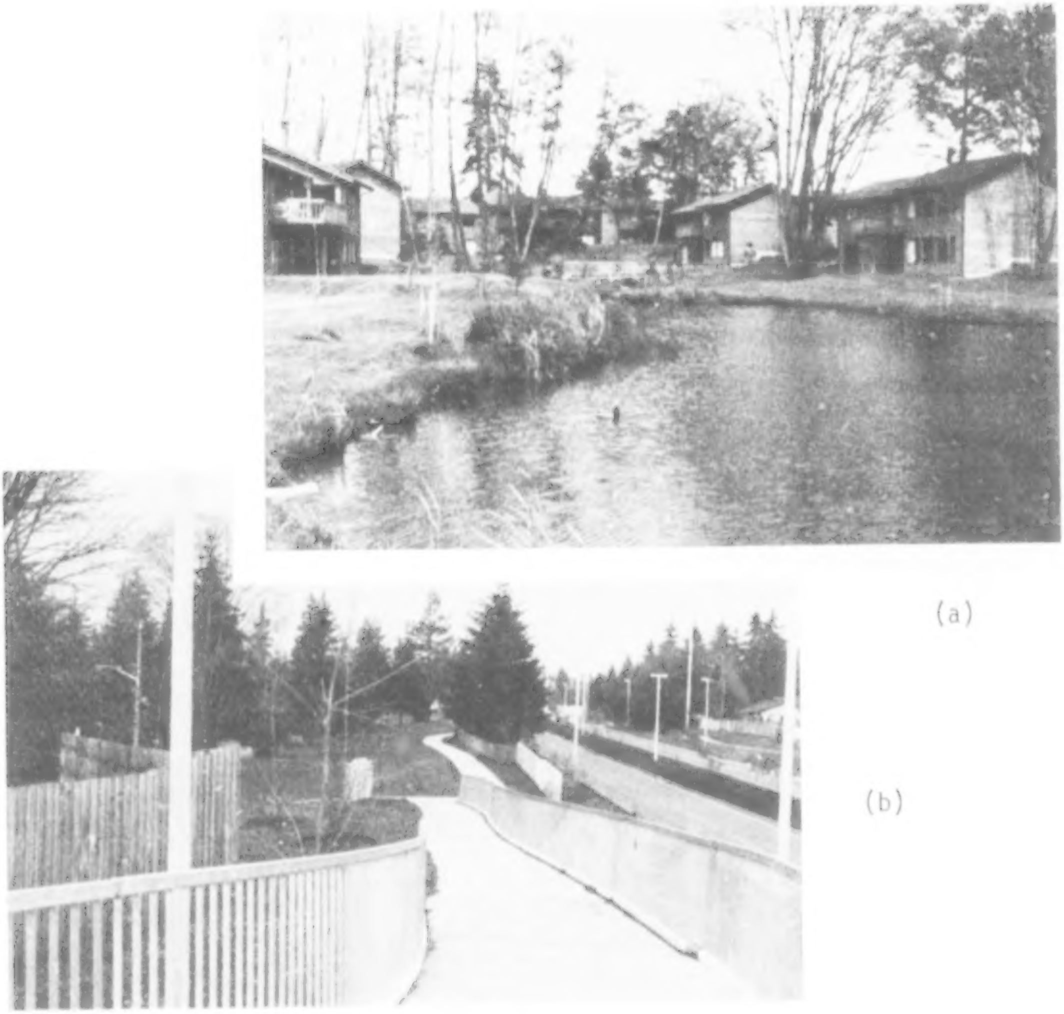


Figure 16. Multiple stormwater facilities: (a) stormwater detention pond integrated into the development landscaping, and (b) detention ponds integrated into a roadside park--ponds are grassed depressions.

# SECTION 5

# SOURCE DETENTION OF URBAN RUNOFF MONTGOMERY COUNTY, MARYLAND

Montgomery County and the region surrounding Washington, D.C., have been using source controls to limit urban runoff. In 1965, initial sediment controls were established to correct erosion problems, and state and local ordinances setting basic control requirements were developed and adopted in the early 1970s. Montgomery County adopted an ordinance in 1971 stating that the 2 year storm shall be stored and released at predevelopment rates. This ordinance applies to all new development.

Over 800 source control facilities have been constructed in Montgomery County. Most are small individual controls--wet and dry detention ponds; underground storage vaults; parking lot and rooftop storage; and infiltration/percolation systems. Detention ponds are the most common control and are used extensively in residential and industrial developments. The detention ponds were primarily developed for construction sediment control and postconstruction volume (flood/drainage) control, but other benefits, including pollution control, recreation, and aesthetics, are now being realized.

The source control strategy in Montgomery County is moving toward larger control areas, 80 to 240 ha (200 to 600 acres), representing basinwide applications. A basinwide management plan and several source detention sites are used as examples to illustrate control for large areas serving several different land uses. These sites have incorporated, where possible, multiuse features.

## REGIONAL APPROACH TO RUNOFF CONTROL

The control of nonpoint sources of pollution in the Washington, D.C., area is becoming increasingly important, particularly as dry-weather treatment levels increase. Many of the existing water quality problems have been traced to urban runoff. As rural areas become developed, the problems of and the potential for flooding, erosion, and sediment deposition in the natural streams draining the region have increased. Montgomery County, by adopting and enforcing source control ordinances, has one of the most advanced programs in the nation.

### Area Characteristics

Montgomery County is northwest of Washington, D.C., and borders on and drains into the Potomac River, as shown in Figure 17. The topography consists of rolling hills with slopes ranging from 5 to 10%. The soils in the uplands are well drained and subject to moderate erosion. The soils along the natural drainage courses are poorly drained and subject to high erosion.

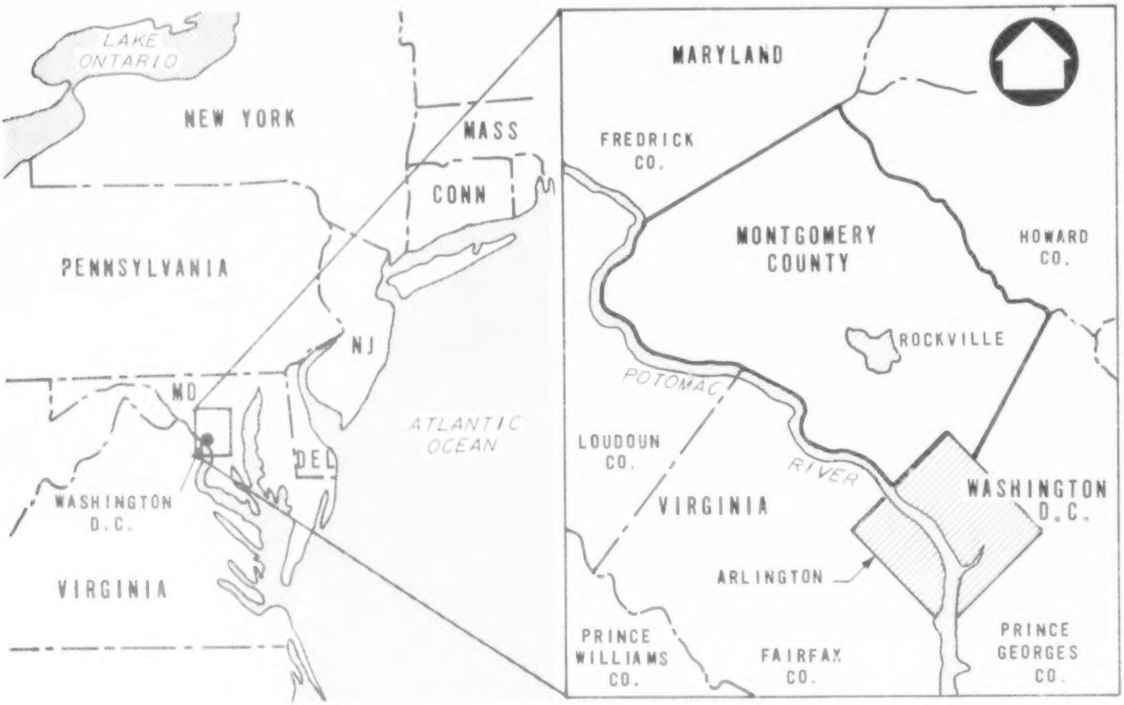


Figure 17. Location of Montgomery County, Maryland, and the surrounding Washington, D.C., area.

The mean annual precipitation in the county is approximately 104 cm (41 in.), with about 57% of the precipitation occurring from April through September. Thunderstorms occur on an average of about 30 days per year and 75 to 80% of these occur during the summer months [1].

Montgomery County has been changing from a rural, agricultural area to an urban area with single-family and high-density residential developments. Commercial and light industrial centers have also been constructed and have also contributed to the change in runoff characteristics of the land. The projected changes in land use for a 5,750 ha (13,200 acre) drainage area in Montgomery County are shown in Table 26. The change in land use, based on the ultimate development capacity using future land use projections and existing zoning, could be representative of the long-range urbanization process in the Washington, D.C., area [2].

Potential urban runoff impacts of development can be associated with the increase in residential land use. Although changes in commercial and industrial land use are only a fraction of the residential increase, equal attention and controls should be provided because these areas are highly impervious and have high use intensities.

Table 26. PROJECTED CHANGES IN LAND USE IN THE WATTS BRANCH DRAINAGE AREA [2]

| Land use category | 1975 land use, acre | Projected long-range land use, acre | Change in total drainage area, % |
|---|---|---|---|
| Total residential | 2,720 | 9,952 | 50.9 |
| Commercial | 135 | 167 | 0.2 |
| Industrial | 401 | 1,498 | 7.7 |
| Institutional | 295 | 435 | 1.0 |
| Open recreation | 774 | 1,883 | 7.8 |
| Agricultural, vacant land | 9,862 | 252 | -67.6 |
| Surface water (ponds) | 21 | 21 | 0 |
| Total | 14,208 | 14,208 | 0 |

## Problem Assessment

The increased urban runoff from developing areas creates both quantity problems and water quality problems. Source controls were originally implemented to limit flooding, erosion, and sediment deposition. However, with increasing water quality goals and standards, source controls have also been recognized as providing pollutant reduction benefits. Eroding soils and pollutants washed off land surfaces are directly influenced by the rate and volume of runoff from the urban area.

### Quantity--

Channel and stream erosion and local flooding have been caused by increased flow from developed and developing areas in Montgomery County. The flow problems are intensified in the smaller, upper drainage areas and are moderated as the size of the drainage area increases. In the upper portions of the Watts Branch watershed, the increase in peak flow from existing development was estimated at over 110% of the natural predevelopment rates for the 2 year storm. Yet, for the entire drainage area, almost 12 times the area of the upper portion, the increase in peak flow for the 2 year storm was estimated at about 10% [2]. These increased flows in the upper watersheds can cause substantial erosion, flooding, sediment deposition, and stream damage.

Erosion from these flows has cut into natural streams and doubled their original predevelopment width, undercut stream banks and structures, and damaged vegetation. Massive sediment yields are washed into receiving waters or are deposited in streams. Erosion losses as high as 3,800 $m^3/km^2 \cdot yr$ (8,000 $yd^3/mi^2 \cdot yr$) have been estimated for the Watts Branch drainage area [2]. Results of stream erosion problems are shown in Figure 18.

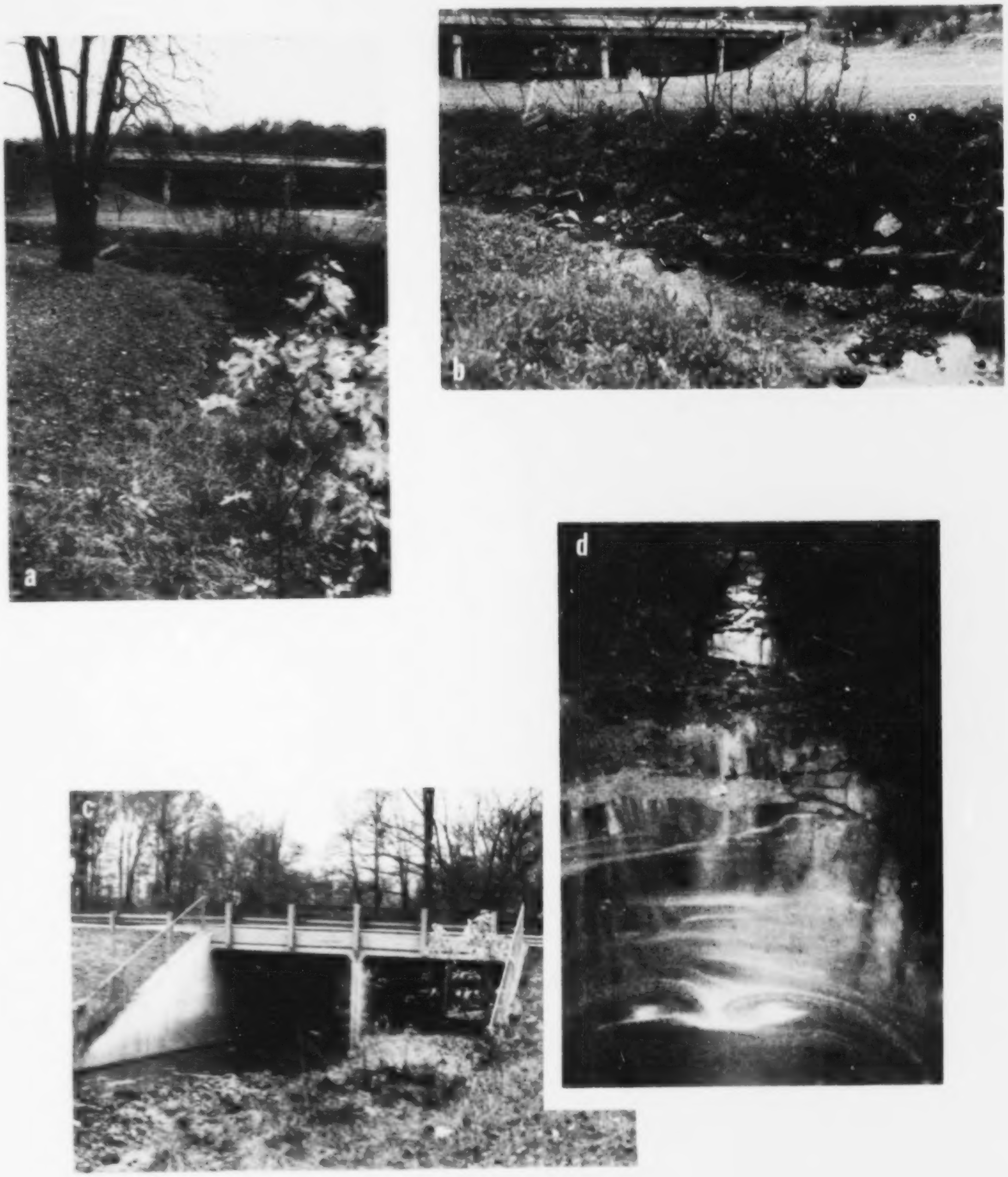


Figure 18. Flow, erosion, and sediment deposition problems: (a) stream subject to increased flow from developed watershed, (b) stream channel over twice natural width--steep eroded banks have been riprapped, (c) sediment deposition of eroded material in half of bridge culvert, and (d) undercut and damaged vegetation.

Quality--

Substantial pollutant loads are associated with urban runoff, and eroded material can potentially contribute most of the suspended solids and phosphorus loads. Storm flows in the 5,750 ha (14,200 acre) Watts Branch drainage area annually contribute over 1,485,000 kg (3,270,000 lb) of suspended solids, or about 88% of the total annual load [2]. Estimated annual storm flow loads for $BOD_5$, nitrogen, and phosphorus are shown in Table 27.

Table 27. ESTIMATED ANNUAL STORM FLOW LOADS IN THE WATTS BRANCH DRAINAGE AREA [2]

| Constituent | Annual storm flow load, lb/yr | Storm flow, % of total annual load[a] | Unit loading, lb/acre·yr |
|---|---|---|---|
| Suspended solids | 3,270,000 | 88 | 230 |
| $BOD_5$ | 240,000 | 86 | 17 |
| Nitrogen | 33,000 | 43 | 2.3 |
| Phosphorus - $PO_4$ | 8,000 | 64 | 0.6 |

a. Total annual load = base flow load + storm flow load.

In the Washington, D.C., area, pollutant concentrations in urban runoff increase slightly for land uses with higher impervious areas or a higher intensity of use. Commercial land use, as shown in Table 28, generally has the highest concentrations, particularly for lead and zinc.

Table 28. POLLUTANT CONCENTRATIONS IN URBAN RUNOFF FOR SEVERAL LAND USES IN THE WASHINGTON, D.C., AREA [3, 4]

| | Pollutant concentration, mg/L | | | | |
|---|---|---|---|---|---|
| Land use | COD | Total nitrogen | Total phosphorus | Lead | Zinc |
| Low density residential | 70-120 | 2-4 | 0.3-0.4 | 0.05-0.1 | 0.02-0.1 |
| Medium density residential | 80-130 | 2-3 | 0.3-0.4 | 0.1-0.2 | 0.05-0.2 |
| High density residential | 70-90 | 2-3 | 0.3-0.5 | 0.1-0.2 | 0.05-0.3 |
| High rise residential | 50-100 | 1-2 | 0.2-0.3 | 0.1-0.2 | 0.1-0.2 |
| Commercial | 90-120 | 2-3 | 0.2-0.3 | 0.3-0.5 | 0.1-0.4 |

Pollutant loads vary significantly between different land uses with different impervious characteristics. Generally, land uses in high impervious areas, such as high density residential and commercial, yield the highest runoff volume and the highest pollutant load. The mean and the range of pollutant loads for sampled urban runoff are shown in Table 29. The loads are presented on a daily basis and represent discrete storm loadings.

Table 29. STORM POLLUTANT YIELDS FOR SEVERAL LAND USES IN THE WASHINGTON, D.C., AREA [5]

| | | Pollutant yield, lb/acre·d | | | | | |
|---|---|---|---|---|---|---|---|
| | Number of storms | Suspended solids | | Total nitrogen | | Total phosphorus | |
| Land use | | mean | range | mean | range | mean | range |
| Low density residential | 23 | 15 | 0-110 | 0.2 | 0-0.7 | 0.04 | 0-0.2 |
| Medium density residential | 42 | 21 | 1-270 | 0.7 | 0.1-5.1 | 0.12 | 0.01-0.8 |
| High density residential | 64 | 31 | 0-190 | 0.6 | 0-2.9 | 0.11 | 0-0.5 |
| High rise residential | 21 | 18 | 0-120 | 0.8 | 0-3.7 | 0.10 | 0-0.4 |
| Commercial | 50 | 43 | 0-320 | 1.0 | 0-6.5 | 0.12 | 0-0.8 |
| Rural | 6 | 29 | 0-170 | 0.1 | 0-0.3 | 0.01 | 0-0.1 |
| Agricultural | 32 | 83 | 0-620 | 0.6 | 0-4.9 | 0.18 | 0-2.1 |

## Source Control Approach - Regulatory Requirements

The regulatory objective of source controls in Montgomery County, originating with the SCS concepts of mitigating increased flooding, sedimentation, land erosion, and accelerated streambank erosion, is to limit urban runoff to natural predevelopment rates and volumes. Water quality benefits from the control of urban runoff have also been recognized. Through a program of regulatory requirements, a large number of source controls have been constructed and design concepts have been improved.

All new development and construction, which increase the impervious area, must have stormwater management facilities. These facilities can include onsite controls, those located on the site being controlled, or offsite controls, those that are located downstream from the area to be controlled and can control a number of developments. Offsite controls, generally located in a stream or drainage valley, are designed to control subwatersheds or whole drainage areas and are used in place of many small onsite controls.

Certain exemptions are allowed under the regulatory policy for development sites having minimal land disturbance or small percentages of impervious area. Waiver applications for detached single family residential developments may be allowed for the following minimum lot sizes within certain subdivision sizes [6]:

| Minimum lot size | Maximum subdivision size |
|---|---|
| 1 acre | 10 acres |
| 0.5 acre | 5 acres |
| 15,000 $ft^2$ | 2 acres |
| 9,000 $ft^2$ | 2 acres |
| 6,000 $ft^2$ | 2 acres |

Exemptions are also considered for multifamily residential, industrial, commercial, and institutional developments meeting developed area and impervious area limitations. Developments that expand beyond the limits set by a waiver would be required to provide stormwater management controls for the entire site.

The regulations require the control or storage of stormwater runoff in excess of the natural predevelopment flow from the 2 year storm. Other than requiring source controls, there is currently no mechanism for enforcement of maintenance once the facilities are built. The role of the county in enforcing the stormwater management policy is limited to design review and approval, permit issuance, and inspection during construction. On private land, the owner must maintain the control facility.

### Implemented Controls

A large number of stormwater management controls are used in Montgomery County, including detention ponds, underground storage vaults, parking lot storage, rooftop storage, and infiltration systems. Over 42% of all controls used are detention ponds, 69% of which are associated with residential developments. A number of infiltration systems have also been used, but principally for institutional, commercial, and industrial developments. These can include detention/infiltration ponds and infiltration pits (dry wells). A summary of the distribution of 832 control measures by type and land use is presented in Table 30.

Table 30. SUMMARY OF STORMWATER CONTROLS BY LAND USE IN MONTGOMERY COUNTY [7]

| Land use | Total number of controls | Stormwater controls, % of total | Distribution by type of control, % | | | | |
|---|---|---|---|---|---|---|---|
| | | | Detention pond | Underground storage | Parking lot storage | Rooftop storage | Infiltration system |
| Residential | 256 | 31 | 69 | 3 | 3 | 9 | 16 |
| Institutional | 274 | 33 | 26 | 10 | 2 | 14 | 48 |
| Commercial | 112 | 13 | 25 | 8 | 9 | 20 | 38 |
| Industrial | 93 | 11 | 32 | 8 | 13 | 25 | 22 |
| Unclassified | 97 | 12 | 42 | 11 | 4 | 6 | 36 |
| Total | 832 | 100 | 42 | 7 | 4 | 14 | 33 |

Most of the controls have been constructed on development sites of less than 4 ha (10 acres). Of the planned controls shown in Table 31, the most widely used controls for developments of 0.4 to 2 ha (1 to 5 acres) are infiltration systems, underground storage vaults, dry detention/sedimentation ponds, swales, rooftop storage, and parking lot storage. Detention ponds are used, almost exclusively, for larger control areas, and dry ponds represent the largest fraction of controls used for all drainage area sizes.

### Table 31. SUMMARY OF PLANNED STORMWATER CONTROLS BY DRAINAGE AREA SIZE IN MONTGOMERY COUNTY [6]

| Type of control | Total number of controls | Stormwater controls, % of total | Number of controls by drainage area category, acres | | | | | |
|---|---|---|---|---|---|---|---|---|
| | | | 1-5 | 6-10 | 11-50 | 51-100 | 101-500 | >500 |
| Infiltration systems | 47 | 24 | 47 | -- | -- | -- | -- | -- |
| Underground vaults | 29 | 15 | 29 | -- | -- | -- | -- | -- |
| Dry pond | 53 | 28 | 20 | 11 | 16 | 4 | 1 | 1 |
| Wet pond | 10 | 5 | -- | -- | 4 | 3 | 1 | 2 |
| Underground pipe | 9 | 5 | 6 | 2 | 1 | -- | -- | -- |
| Swales | 19 | 10 | 15 | 3 | -- | 1 | -- | -- |
| Rooftop storage | 16 | 8 | 16 | -- | -- | -- | -- | -- |
| Parking lot storage | 10 | 5 | 10 | -- | -- | -- | -- | -- |
| Total | 193 | 100 | 143 | 16 | 21 | 8 | 2 | 3 |
| Controls by area, % | -- | -- | 74 | 8 | 11 | 4 | 1 | 2 |

Some of the facilities under construction will serve larger drainage areas or several developments to reduce costs and maintenance problems. Larger facilities may realize some economies of scale, where small systems, less than 4 ha (10 acres), would not. Most of the larger facilities being constructed are detention ponds with permanent pools, which have better pollutant reduction capabilities in addition to flow control. The permanent pool volume acts as a sink that traps pollutants from the stormwater flows, providing increased removals because of the long detention times in the pool. An example of areawide source control planning and examples of individual detention facilities in Montgomery County that represent the most promising source control technology are discussed in the following.

#### Watts Branch Management Plan--

Three alternative stormwater management concepts were evaluated to control urban runoff from the 5,750 ha (14,200 acre) Watts Branch drainage area. These included (1) offsite headwater or tributary small-scale detention facilities; (2) onsite detention for individual developments; and (3) land use control within the 100 year flood plain, with no structural controls [2].

The offsite detention facilities would include a system of small-scale ponds from 0.8 to 4.0 ha (2 to 10 acres) in size, designed to control flowrates and volumes from the 2, 10, and possibly 100 year storm, at predevelopment levels. The detention pond system would have permanent pools to increase pollutant removal. This type of detention can be used not only to control runoff from developing sites, but also to control runoff from developed areas.

Although onsite detention facilities can control peak flowrates for a design frequency storm, usually a 2 year storm, they fall short of their intended efficiency with flows from any storms different than the design frequency.

Also, onsite detention is applicable only to developing sites; site conditions, such as available area, size requirements, soils, or topography, can limit the effectiveness of onsite controls. Most onsite controls are dry ponds that are used to reduce erosion and sedimentation problems. Maintenance responsibilities are usually passed on by the developer to the owner, who may or may not maintain the controls.

The recommended plan for the Watts Branch drainage area involves portions of all three alternatives, but the principal effort is contructing small-scale offsite detention ponds to control flows up to the 100 year storm. Where offsite controls are not feasible, developers would construct onsite controls within easements dedicated to the local agency or the county for maintenance. The plan also recommends that the county continue to acquire flood plain land for parks and prohibit filling or construction within the 100 year flood plain [2].

The required facilities for onsite and offsite alternative plans, together with estimates of total costs and expected annual benefits, are compared in Table 32. Although the costs of offsite controls are slightly higher than the costs of onsite controls, the expected annual benefits are greater for offsite controls.

Table 32. COMPARISON OF ONSITE AND OFFSITE CONTROLS FOR THE WATTS BRANCH DRAINAGE AREA [2]

| Parameter | Alternative<br>Onsite flow control | <br>Offsite flow and quality control |
|---|---|---|
| Drainage area controlled, acre | 2,960 | 2,900 |
| Storage | | |
| Flowrate control storage, $10^6$ $ft^3$ | 6.97 | 9.19 |
| Water quality control storage, $10^6$ $ft^3$ | 0 | 2.66 |
| Total storage, $10^6$ $ft^3$ | 6.97 | 11.85 |
| Estimated number of structures | 105 | 6 |
| Estimated land requirement, acres | 80 | 72 |
| Annual costs[a] | | |
| Annual capital, $[b] | 907,000 | 1,050,000 |
| Operation and maintenance, $ | 125,000 | 84,000 |
| Total annual costs, $ | 1,032,000 | 1,134,000 |
| Annual benefits, $[c] | 86,700 | 346,700 |
| Net annual cost, $ | 945,300 | 787,300 |

a. ENR 3000.

b. Capital costs include construction, land, and design, annualized at 7% interest for 10 years.

c. Includes savings in water treatment plant costs, land loss and sediment damage, damage to vegetation, and cleanup.

Source Detention Facilities--

A number of individual detention facilities are planned or are being constructed in Montgomery County. Most of these facilities are offsite controls; however, some onsite controls are also being constructed.

Crabbs Branch Facilities--In the Crabbs Branch drainage area, two detention facilities are being constructed: an onsite detention pond to control stormwater runoff from a complex of county government warehouses and maintenance depots and an offsite detention pond to provide overall control for the developing watershed.

The onsite detention pond provides sediment control for an 18 ha (45 acre) county service park, still under construction. The pond is a rectangular basin, 122 m by 43 m (400 ft by 140 ft) with a pool surface area of about 0.5 ha (1.3 acres), as shown in Figure 19. Runoff enters the pond at two points through reinforced-concrete drains. The basin outlet is a 1.2 m (48 in.) diameter, nonperforated corrugated metal pipe riser and barrel. The 1.8 m (6 ft) high riser is topped with a closed lid and an antivortex hood equipped with a trash rack to retain debris and other floating material. The riser also has a 10.2 cm (4 in.) hole to maintain the pool level 0.46 m (1.5 ft) below the crest of the riser. A grassed overflow channel is also provided as an emergency spillway. The basin will be retained as a permanent flow control pond after the service park is completed.

Figure 19. Onsite sediment detention pond for developing county service park.

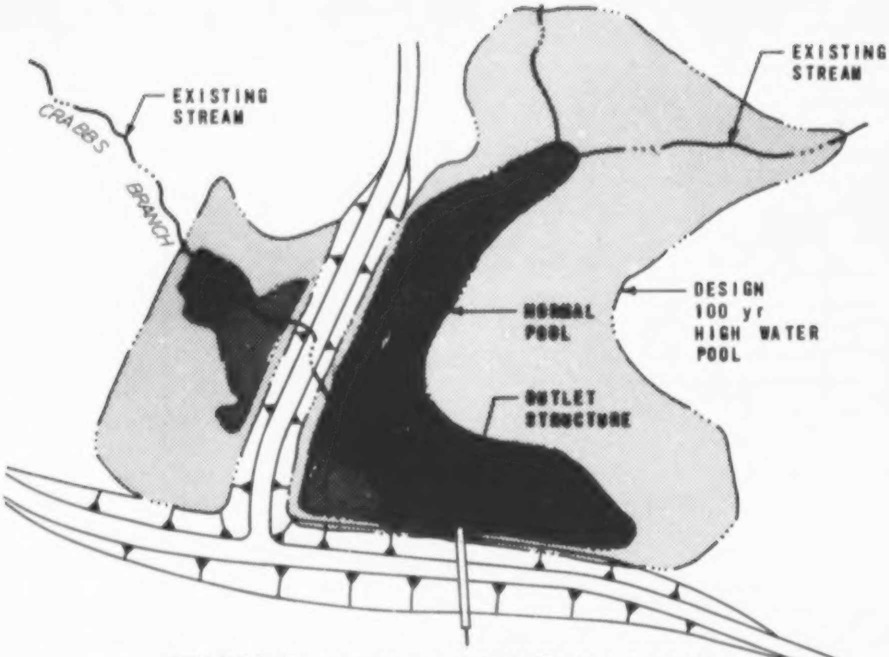
Downstream from the service park, an offsite detention pond will be constructed to control up to the 100 year storm flows from a 238 ha (590 acre) tributary area. The facility includes an earthfill dam, a permanent pool to collect sediment and other settleable pollutants, a principal spillway, and an outlet structure, as shown in Figure 20. The detention pond is expected to reduce peak stormwater runoff discharge rates to or below estimated 1973 levels for the 2 year, 3 hour; 10 year, 3 hour; and the 100 year, 24 hour storm events. The spillway and outlet structure were also designed to convey the Maximum Probable Flood without overtopping the dam. The design features of the detention facility are described in Table 33.

Table 33. DESIGN FEATURES OF THE CRABBS BRANCH OFFSITE DETENTION FACILITY [8]

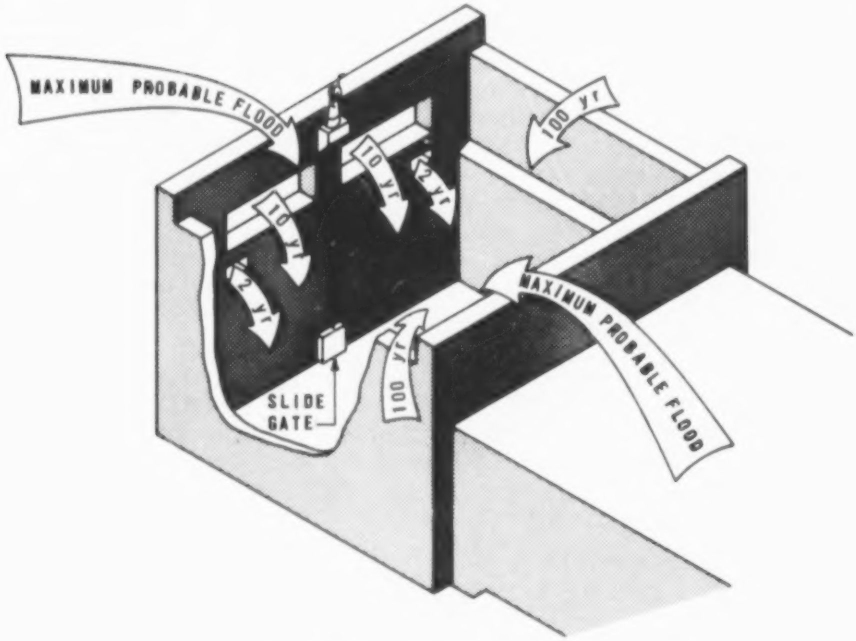
| Description | Value |
|---|---|
| Watershed area, acres | 590 |
| Storage pool | |
| Permanent pool area, acres | 6.8 |
| Permanent pool volume, $10^6$ ft$^3$ | 1.4 |
| Design storage pool area, acres | 21.0 |
| Design storage volume, $10^6$ ft$^3$ | 7.5 |
| Outlet structure | |
| Riser dimensions, ft | 22.0 x 20.0 |
| Riser height, ft | 19 |
| Orifice size (2 yr flow) 2 each, ft | 2.17 x 1.0 |
| Orifice size (10 yr flow) 2 each, ft | 16.0 x 3.0 |
| Weir length (100 yr flow) 2 each, ft | 20.0 |
| Weir length (MPF) 2 each, ft[a] | 22.0 |
| Outlet conduit size, ft | 9.5 x 9.5 |
| Outlet length, ft | 145 |

a. Maximum probable flood.

Montgomery Mall Lake--An offsite stormwater storage/detention pond was constructed to control a 60 ha (148 acre) subwatershed of Cabin John Creek in Montgomery County. The drainage area includes the impervious area of a large shopping mall, several apartment complexes, townhouses, a major highway, and several secondary roads.



a. PROPOSED OFFSITE DETENTION BASIN



b. DETAIL OF OUTLET STRUCTURE

Figure 20. Proposed Crabbs Branch detention basin to control up to the 100 year storm flow [8].

Drainage from the area enters the pond at six different locations, with the major inflow point serving the shopping mall and draining about 38 ha (95 acres) of the watershed. This overflow point enters the head end of the pond through a 1.7 m (66 in.) pipe.

The pond has a 2.4 ha (5.9 acre) permanent pool, about 229 m by 107 m (750 ft by 350 ft), and is used to limit peak storm flows and to reduce pollutants. The level of the pool, which has about 45,600 $m^3$ (1,600,000 $ft^3$) of dead storage capacity, can vary from 0.9 to about 4 m (3 to 13 ft). The primary outflow is controlled by a 0.6 m (24 in.) corrugated metal pipe riser, topped with a 0.9 m (36 in.) hood. The emergency overflow is controlled by a 12 m (40 ft) grassed overflow channel and a concrete, riprapped crest about 0.9 m (3 ft) above the riser crest [9]. The detention pond, shown in Figure 21, is designed to control an inflow of 0.6 $m^3/s$ (22 $ft^3/s$) and to release the volume at about 0.06 $m^3/s$ (2 $ft^3/s$).

Wheaton Branch Facility--An offsite stormwater detention pond is being constructed to control runoff from a 314 ha (775 acre), totally developed area on the Wheaton Branch in Montgomery County [10]. The detention facility, a dry pond, will be drained following a storm. The design flood storage will cover about 6.3 ha (15.5 acres) and will have a total storage capacity of about 138,000 $m^3$ (4,880,000 $ft^3$). The pond will control flows from the 2 to 100 year storm. A summary of the expected operation of the facility for various storm frequencies is presented in Table 34. The facility is shown in Figure 22.

### Design Considerations

The two different strategies for controlling urban runoff by detention, onsite and offsite storage, are designed to control at least the peak flowrates of the 2 year storm to meet regulatory standards and prevent downstream erosion and sediment deposition. Most large offsite controls are also designed to control several different, more extreme runoff events, up to the 100 year storm flow, while most small onsite controls are designed for a 2 year return period. Designing for one return period storm may not provide the desired level of control for return period storm flows that are different from the design storm.

The results of a mathematical model simulation to evaluate how various control designs affect the flows from different return period storms are shown in Table 35. The values represent a control design--the postdevelopment peak discharge for a given return period storm equals the release rate for the specified predevelopment return year storm. The impacts of different storm frequencies other than the design frequency are also shown in Table 35.

Other elements that affect stormwater control performance and that should be considered in design include:

- Storm duration
- Control facility maintenance
- Soil characteristics
- Downstream velocity and channel erosion potential

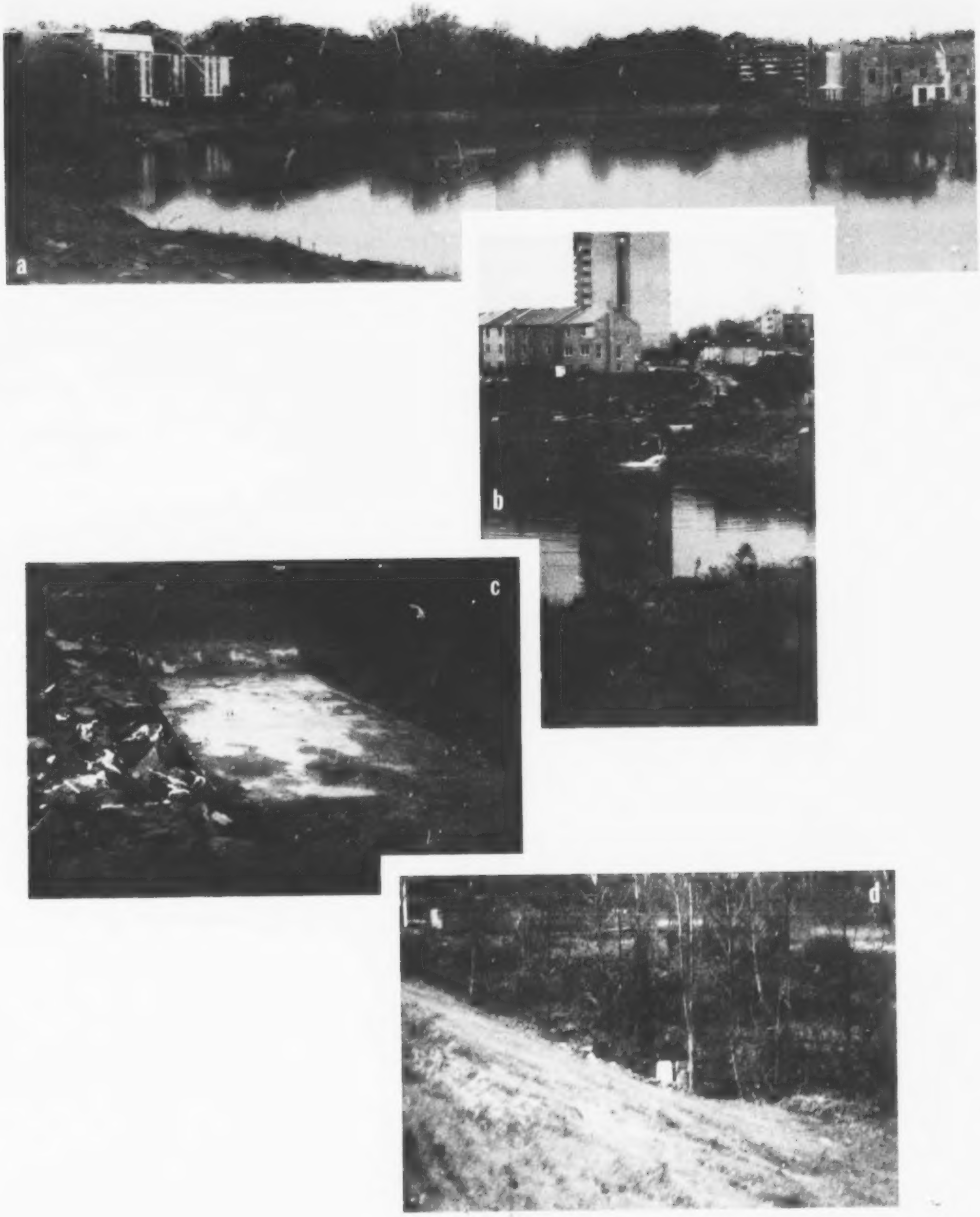


Figure 21. Montgomery Mall Lake offsite storage/detention pond: (a) permanent pool lake with dam in background, (b) principal 66 in. inflow pipe at head end of lake, (c) emergency overflow spillway, and (d) pond overflow point to receiving stream equipped with a flow recorder.

Table 34. EXPECTED HYDRAULIC OPERATION OF THE WHEATON BRANCH OFFSITE, DRY DETENTION POND

| Storm frequency | Pond surface area, acres | Storage |  | Peak flow, ft$^3$/s |  |
|---|---|---|---|---|---|
|  |  | ft$^3$ | Watershed, in. | Inflow | Outflow |
| 2 yr-24 hr | 4.9 | 620,000 | 0.22 | 840 | 510 |
| 5 yr-24 hr | 6.9 | 1,440,000 | 0.51 | 1,270 | 590 |
| 10 yr-24 hr | 7.7 | 2,220,000 | 0.79 | 1,680 | 820 |
| 50 yr-24 hr | 7.9 | 2,620,000 | 0.93 | 2,250 | 1,100 |
| 100 yr-24 hr | 8.5 | 3,250,000 | 1.16 | 2,690 | 1,930 |
| Design storm[a] | 15.5 | 4,880,000 | 1.73 | 4,050 | 2,790 |

[a] An 11 in. rainfall in 6 hours.

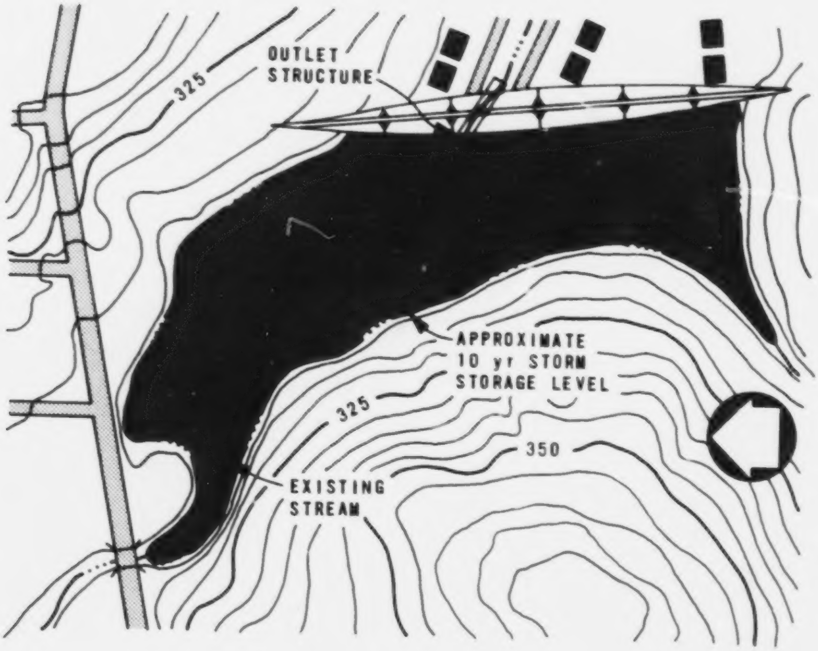


Figure 22. Proposed plan of the Wheaton Branch offsite dry detention pond.

Table 35. RESPONSE OF PEAK STORM FLOW THROUGH A DETENTION SYSTEM DESIGNED FOR SPECIFIC RETURN PERIODS [11]

| | Storm flow return period, yr | | |
|---|---|---|---|
| Level of control/design | 2 | 10 | 100 |
| Predevelopment flow, ft$^3$/s | 9 | 29 | 77 |
| Post development flow through design control, ft$^3$/s | | | |
| 2 yr storm released at 2 yr predevelopment rate | 9 | 22 | 42 |
| 10 yr storm released at 10 yr predevelopment rate | 14 | 29 | 57 |
| 10 yr storm released at 2 yr predevelopment rate | 6 | 9 | 27 |
| 100 yr storm released at 100 yr predevelopment rate | 15 | 33 | 77 |

The design approach evaluating stormwater detention controls should consider all of these elements to more closely anticipate how a control will respond to a storm. Several design approaches were evaluated by comparing their accuracy, training requirements, relative design costs, and applicability of using or determining the effects of the design elements previously listed; the comparison is given in Table 36.

Table 36. COMPARISON OF HYDROLOGIC DESIGN APPROACHES FOR STORMWATER DETENTION FACILITIES [11]

| | Design approach | | | | |
|---|---|---|---|---|---|
| | | | | Conceptual models | |
| Evaluation criteria | Graphical | Empirical | Unit-hydrograph | Simple | Complex |
| Design elements | | | | | |
| Multiple return period | No | Maybe | Yes | Yes | Yes |
| Storm duration | No | Maybe | Yes | Yes | Yes |
| Maintenance | No | No | No | Maybe | Yes |
| Soil characteristics | No | No | Maybe | Yes | Yes |
| Downstream effects | No | No | Maybe | Yes | Yes |
| Accuracy | Low | Low | Medium | High | High |
| Training requirements | Low | Low | Medium | Medium | High |
| Relative design cost | Low | Low | Medium | Medium | High |

The method, outlined in Urban Hydrology for Small Watersheds, Technical Release No. 55, 1975, by the SCS, is used in Montgomery County and is an example of a simple conceptual model [12].

Montgomery County uses a solid riser design rather than a perforated riser design to maintain a permanent pool to maximize pollutant removal. Studies indicate that peak flow reduction is a function of the riser characteristics (height and size), and that sediment and pollutant trapping is affected by the size and shape of the pool [13].

## PERFORMANCE OF STORMWATER DETENTION PONDS

Stormwater detention ponds, originally used to reduce peak storm flowrates in downstream segments, also reduce pollutants. Although performance data are limited, some information on efficiency has been compiled and used in predicting system response to various storm durations and frequencies.

### Peak Flow Reduction

Both onsite and offsite detention ponds reduce peak storm flows. Flow reduction efficiencies can approach about 90% for flows at or less than the design storm flow.

Thirty-six storms were monitored during 1977 at the onsite detention pond serving the Montgomery County service park. Peak flow reduction was consistently about 90% for the small volume, short duration storms. Peak flow reduction dropped to about 60% for a larger volume, longer duration storm flow that peaked at 3.3 $m^3/s$ (177 $ft^3/s$) [13].

The expected operating conditions for the Crabbs Branch offsite detention pond are summarized in Table 37. These larger facilities using multiple stage risers can reduce peak flows from multiple return period storms to or below base or predevelopment levels.

Table 37. SUMMARY OF EXPECTED FLOW REDUCTION PERFORMANCE FOR THE CRABBS BRANCH OFFSITE DETENTION POND [8]

| | Peak discharge, $ft^3/s$ | | | | |
|---|---|---|---|---|---|
| Flood flow frequency, yr | 1973 flow conditions | Projected condition without detention | Projected condition with detention | Pool elevation above permanent pool elevation, ft | 1973 discharge, % |
| 2 | 273 | 892 | 269 | 4.7 | 99 |
| 10 | 600 | 1,591 | 439 | 6.4 | 73 |
| 100 | 1,139 | 2,623 | 913 | 8.5 | 80 |
| Maximum Probable Flood | 5,180 | 5,793 | 4,234 | 15.0 | 82 |

### Pollutant Trap Efficiencies

Pollutant trap efficiencies were monitored at the Montgomery Mall Lake detention facility and are presented in Table 38. The median trap efficiency values suggest that pollutant removal can be high if the basin is properly designed for increased pollutant removal, i.e., having large permanent pool storage volume.

### Table 38. MEDIAN POLLUTANT TRAP EFFICIENCIES, MONTGOMERY MALL LAKE OFFSITE DETENTION FACILITY [9]

| Parameter | Units | Inflow rate | Outflow rate | Trap efficiency, % |
|---|---|---|---|---|
| $BOD_5$ | $10^{-3}$ lb/s | 11.4 | 0.4 | 97 |
| $BOD_{20}$ | $10^{-3}$ lb/s | 19.7 | 1.4 | 93 |
| COD | $10^{-3}$ lb/s | 60.5 | 2.1 | 97 |
| TOC | $10^{-3}$ lb/s | 13.2 | 0.5 | 96 |
| Orthophosphate | $10^{-3}$ lb/s | 0.3 | 0.02 | 93 |
| Total phosphorus | $10^{-3}$ lb/s | 0.7 | 0.007 | 99 |
| Ammonia - N | $10^{-3}$ lb/s | 6.8 | 0.02 | 99 |
| Zinc | $10^{-6}$ lb/s | 0.5 | 0.003 | 99 |
| Cadmium | $10^{-6}$ lb/s | 0.4 | 0.006 | 98 |
| Lead | $10^{-6}$ lb/s | 0.4 | 0.02 | 96 |
| Iron | $10^{-6}$ lb/s | 10.3 | 0.4 | 96 |

Sediment trap efficiencies for the Montgomery County service park facility averaged better than 92%. Smaller storms produced better removals, but no monitored storms produced less than 88% [13]. A determinisitic model was used to evaluate the effects of different storm durations and frequencies, using a nine storm data set monitored at the service park facility. The predicted pollutant trap efficiencies of the detention basin for the 2 and 10 year storms are summarized in Table 39.

### Table 39. PREDICTED TRAP EFFICIENCIES FROM STORMS OF VARYING DURATION AND FREQUENCY [11]
Percent

| | 2 yr storm duration, h | | | | 10 yr storm duration, h | | | |
|---|---|---|---|---|---|---|---|---|
| Parameter | 0.5 | 1.0 | 2.0 | 6.0 | 0.5 | 1.0 | 2.0 | 6.0 |
| $BOD_5$ | 91 | 88 | 86 | 84 | 86 | 81 | 79 | 80 |
| $BOD_{20}$ | 59 | 57 | 49 | 37 | 54 | 45 | 37 | 27 |
| COD | 76 | 67 | 57 | 48 | 57 | 45 | 37 | 36 |
| TOC | 84 | 80 | 75 | 70 | 76 | 68 | 64 | 63 |
| Orthophosphate | 69 | 65 | 56 | 40 | 62 | 50 | 41 | 29 |
| Total phosphorus | 22 | 17 | 15 | 7 | 17 | 13 | 10 | 3 |
| Ammonia - N | 92 | 90 | 88 | 89 | 85 | 77 | 76 | 82 |
| Zinc | 98 | 97 | 96 | 94 | 96 | 95 | 94 | 93 |
| Cadmium | 79 | 77 | 70 | 55 | 76 | 73 | 68 | 55 |
| Lead | 97 | 97 | 96 | 94 | 97 | 96 | 96 | 94 |
| Iron | 92 | 90 | 87 | 81 | 89 | 85 | 83 | 80 |

COSTS

Most onsite controls are provided by the developer during construction of the development, and the capital costs, including land costs, can be passed on to the eventual owners of the development. The recent directions of Montgomery County's stormwater management program toward larger offsite tributary controls enable developers to contribute to the cost or construction of the offsite facility that would control runoff from the development. Offsite controls are less expensive and are easier to maintain than many privately owned, small structures. A planning study for the Watts Branch drainage area includes six offsite detention ponds and has evaluated the control costs. Individual cost estimates are available for several planned facilities, and the SCS has developed cost guidelines and estimating procedures.

## Watts Branch Management Plan Costs

The total capital cost to provide offsite stormwater controls for approximately 1,175 ha (2,900 acres) of the Watts Branch drainage area is about $7,375,000 (ENR 3000). The capital cost estimates include construction; land acquisition; and planning, design, and supervision, as summarized in Table 40. The average cost for the controlled area is about $6,300/ha ($2,500/acre), and the storage cost is about $\$22/m^3$ ($\$0.62/ft^3$) of total storage capacity, basinwide [2]. The estimated operation and maintenance costs are about $84,000/yr, or about $72/ha·yr ($29/acre·yr) of controlled area.

Table 40. ESTIMATED BASINWIDE COSTS of OFFSITE STORMWATER DETENTION FOR THE WATTS BRANCH DRAINAGE AREA[a] [2]

| Cost component | Cost, $ |
|---|---|
| Construction | 4,195,000 |
| Land[b] | 2,128,000 |
| Planning, design, and supervision | 1,052,000 |
| Total capital costs | 7,375,000 |

a. ENR 3000.

b. Estimate of land is at full value (about $30,000/acre) even though much of the land is publicly owned.

## Offsite Detention Facility Costs

The capital costs of the Crabbs Branch and the Wheaton Branch detention facilities are summarized in Table 41. The costs are present estimated program costs that include planning, design, supervision, land, site improvements and utilities, and construction.

Table 41. ESTIMATED CAPITAL COSTS FOR OFFSITE STORMWATER DETENTION FACILITIES[a] [14]

| Facility location | Service area, acres | Storage capacity, $10^6$ ft$^3$ [b] | Costs, \$1,000 | | | | | Service area cost, \$/acre | Storage cost, \$/ft$^3$ |
|---|---|---|---|---|---|---|---|---|---|
| | | | Design | Land | Site improvements and utilities | Construction | Total | | |
| Crabbs Branch | 590 | 7.54 | 325 | 635 | 62 | 1,166 | 2,188 | 3,700 | 0.29 |
| Wheaton Branch | 775 | 4.88 | 162 | -- | 68 | 818 | 1,048 | 1,350 | 0.21 |

a. ENR 3000.

b. Storage in excess of permanent pool capacity.

## Cost Estimating

Unit cost estimates for stormwater detention ponds in Montgomery County were used by the SCS to develop a first-cut method of estimating costs of onsite stormwater detention facilities. A graphical presentation of the estimated costs is shown in Figure 23.

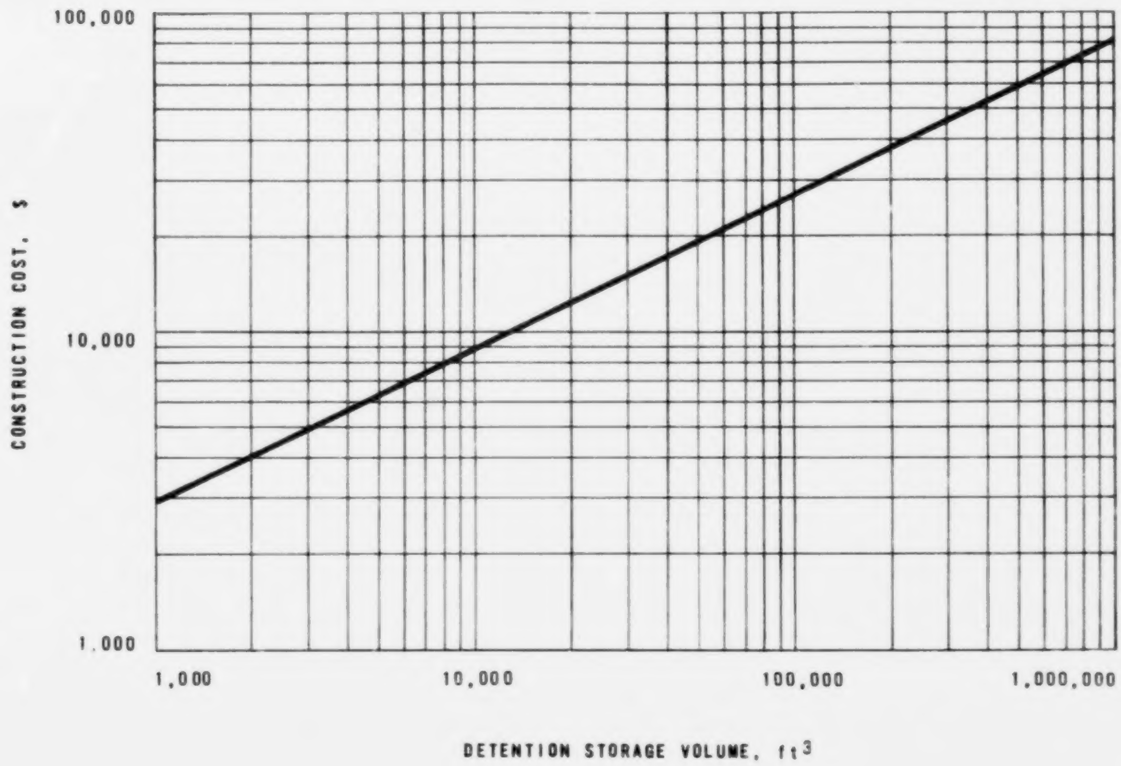


Figure 23. Onsite stormwater detention pond cost curve, ENR 3000 [6].

Several variables were used in a statistical analysis to develop the costs; these include (1) controlled drainage area, (2) storage volume, (3) percentage of impervious area, (4) cost/impervious area, and (5) cost/unit storage volume. There was no strong linear relationship between any of the variables; however, there was a stong curvilinear relationship ($r^2 = 0.856$) between storage volume ($V_s$) and cost/storage volume. These were used to develop the following equation for predicting costs [6]:

$$\$/ft^3 = 106\ V_s^{-0.517}$$

(Equations are adjusted to represent ENR 3000 costs.)

Total costs can be estimated by rearranging this equation:

$$\$ = 106\ V_s^{+0.483}$$

## IMPACTS

With the exception of erosion and sediment deposition, few relationships between stormwater runoff and adverse environmental impacts have been supported by data collection programs. Socioeconomic impacts may actually be more apparent; both onsite and offsite source detention practices have reduced downstream flooding.

### Environmental Impacts

In Montgomery County, runoff from developed and developing areas is transported by receiving streams, and source detention controls potentially reduce both water quantity and quality impacts to this environment. Quantity impacts include (1) flooding; (2) land surface erosion; (3) stream channel erosion; (4) downstream sediment deposition; and (5) reduced groundwater levels and reduced base stream flow resulting from increased runoff from impervious areas and less soil infiltration. Water quality impacts include suspended particles (including sediment), nutrients, toxics, debris, bacteria, and oxygen depletion.

Although no monitoring data are available to quantify the magnitude of the quality impacts to receiving waters, estimates indicate that annual storm flow pollutant loads for BOD and suspended solids are over seven times the annual base flow loads. The impacts of these increased loads are particularly important where the downstream environment or flow is influenced. At a water treatment facility with the water intake influenced by flows from the Watts Branch drainage area, an average additional 21,000 kg/d (46,000 lb/d) of sediment is removed during moderate to heavy storm flows. This condition occurs about 90 d/yr [2].

Environmental impacts of erosion and sedimentation can be seen in stream channels draining developed areas without source controls. Undercut and damaged vegetation, steep eroded stream banks over twice their original width, undermined structures, and deposits of eroded material are evidence of the damage potential. In watersheds where controls are used, environmental damage has been held to a minimum, and streams have retained their natural state.

While many of the environmental impacts of uncontrolled storm runoff are aesthetically undesirable, many also create hazardous conditions and have significant socioeconomic impacts.

## Socioeconomic Impacts

Properly designed detention controls, which consider downstream flows and peak flow timing, can effectively reduce flooding, erosion, and property damage. An estimate of the annual cost of damage to the Watts Branch drainage area from storm flows (without controls) was approximately $430,000 (ENR 3000) [2]. This cost was attributed to the following:

| | |
|---|---|
| Land loss and sediment-generated damages | 27% |
| Cleanup and minor repairs | 2% |
| Personal inconvenience | 1% |
| Additional water treatment | 56% |
| Damage to stream side vegetation | 14% |

Many offsite permanent pool detention ponds are designed to provide multiuse benefits, primarily recreational and aesthetic in nature, as shown in Figure 24. Several detention ponds have restricted access for safety reasons; however, even these ponds can provide aesthetic benefits and are frequently integrated into developments as a part of the overall architectural setting.

Most of the planned offsite detention facilities are located on vacant private land or on publicly owned land and do not require relocation of residences or business. Onsite facilities planned within individual developments require a portion of the developable land; consequently, land costs and the loss of potential profits from the developable land are often passed on in higher costs to the remaining development. Maintenance of the facility also represents a cost to the owner(s) and is often neglected. The movement toward larger offsite facilities lessens the burden of the private developer/owner and potentially reduces the overall unit cost of the control measure.

Proposed funding of offsite controls can come from the county and contributions made by the developer in lieu of providing onsite controls. Depending on the method of county financing, bond or tax, part of the financial burden for the controls may be placed on the public, who benefit from stormwater control.

Public acceptance of offsite multiuse detention ponds is favorable when the pond is integrated into the initial development planning. The pond is considered a selling point, with adjacent lots bringing a premium. However, placing a stormwater detention pond in an existing development has sometimes met with opposition.

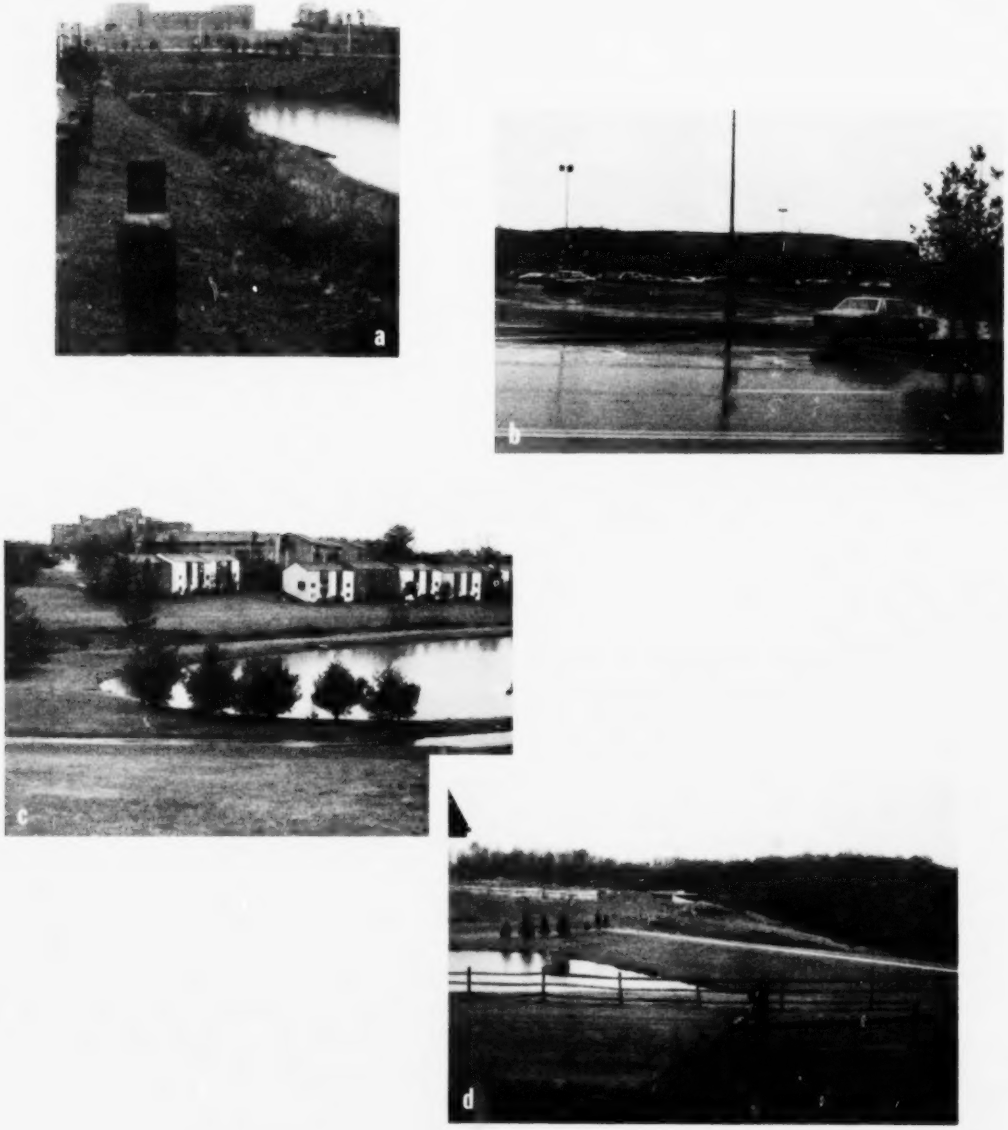


Figure 24. Multiuse stormwater detention facilities, Montgomery County: (a) portion of a detention pond with a permanent pool to detain flows from (b) the highly impervious parking lot of shopping mall; (c) landscaped detention pond integrated into a residential development; and (d) control outlet and dam.

## SECTION 6

## LAND USE PLANNING AND EROSION CONTROL
## LAKE TAHOE, CALIFORNIA

Controlling water quality and protecting the environment of receiving streams and Lake Tahoe from nonpoint sources of pollution are of major concern in the Lake Tahoe Basin. Over the last decade, the increase in population and development in the Tahoe area has resulted in increased nutrient and sediment loadings to Lake Tahoe. Although sanitary sewage is now exported from the basin, land disturbances associated with development continue to create water quality problems and result in increased rates of eutrophication of the lake. Recent planning studies, demonstration projects, and wastewater management programs have focused on controlling these adverse impacts through implementation of BMP technology.

The description of stormwater runoff control using BMPs in the Tahoe area is presented in several parts: (1) the regional approach using the result of the Tahoe Basin 208 planning study to identify general problems, land use planning, and erosion control measures; (2) a site-specific comparison of land use planning practiced in the Tahoe area; and (3) a site-specific description of implemented erosion controls.

Land use planning and erosion/sediment controls can limit the adverse impacts of uncontrolled stormwater runoff. Land use planning, which places controls on sensitive features of the site, can offer distinct economic advantages over correcting existing problems in poorly planned, existing developments.

Comparing the impacts of a well-planned development and a poorly planned development highlights the advantages, effectiveness, and potential socioeconomic impacts of sound land use planning. Erosion/sediment controls can effectively correct the results of poor initial planning or disturbed land area; however, they can have high unit costs compared with the costs of land use planning.

### REGIONAL APPROACH TO RUNOFF CONTROL

The unique features of the Lake Tahoe Basin provide scenic and recreational benefits that attract increasing numbers of people to the area. This growth has stimulated rapid urbanization and development of the shoreline areas and has also created the potential for adverse environmental impacts from nonpoint sources of pollution. The water quality of both the tributary streams and Lake Tahoe is of particular concern because of the increasing rate of pollution from stormwater runoff in these high quality waters.

Recent 208 planning has identified pollutant sources and has proposed countermeasure solutions and planning guidelines to curb uncontrolled development and correct problems in existing developments [1].

## Basin Characteristics

The Lake Tahoe Basin includes about 1,300 $km^2$ (500 $mi^2$) of rugged mountainous terrain; approximately two-thirds of which is in California and one-third in Nevada. Sixty-three major tributary watersheds drain into Lake Tahoe, which covers approximately 40% of the basin area. Approximately 5 to 10% of the basin land area is impervious. The only outlet from the lake is the Truckee River at the north end of the lake. The flushing capacity of the lake is limited. The total surface fluctuation or storage capacity is only 1.8 m (6 ft), which is about 0.6% of the total lake volume, and the estimated mean annual change in storage volume of the lake is only 0.02% of the total volume [2]. The physical characteristics of the Lake Tahoe Basin are summarized in Table 42.

Table 42. PHYSICAL CHARACTERISTICS OF THE LAKE TAHOE BASIN [2]

| | |
|---|---|
| Total surface area of basin, $mi^2$ | 501 |
| Land surface area, $mi^2$ | 310 |
| Lake Tahoe | |
| Surface area, $mi^2$ | 191 |
| Surface elevation, ft | 6,223 to 6,229.1 |
| Length, mi | 22 |
| Width, mi | 12 |
| Length of shoreline, mi | 71 |
| Maximum depth, ft | 1,645 |
| Average depth, ft | 1,027 |
| Storage volume (top 6 ft), acre-ft | 720,000 |
| Total volume | |
| $mi^3$ | 37.43 |
| (acre-ft) | (126,000,000) |

The mean annual precipitation varies from approximately 180 cm (70 in.) in the higher elevations along the western ridge of the basin to about 50 cm (20 in.) on the eastern shoreline area. About 50 to 70% of the precipitation occurs between December and March, in the form of snow. During the summer, the area is subject to highly localized, intense thunderstorms.

The development around the lake is concentrated on the northern and southern shorelines, with scattered development along the western shoreline. Most of the recent development in the basin is related to tourist and recreational activities, which attract over 15 million people annually [1]. The total land area zoned for development is 18%. The existing development is only 12% of the total basin land area [2]. The physical setting and developed areas are shown in Figure 25.

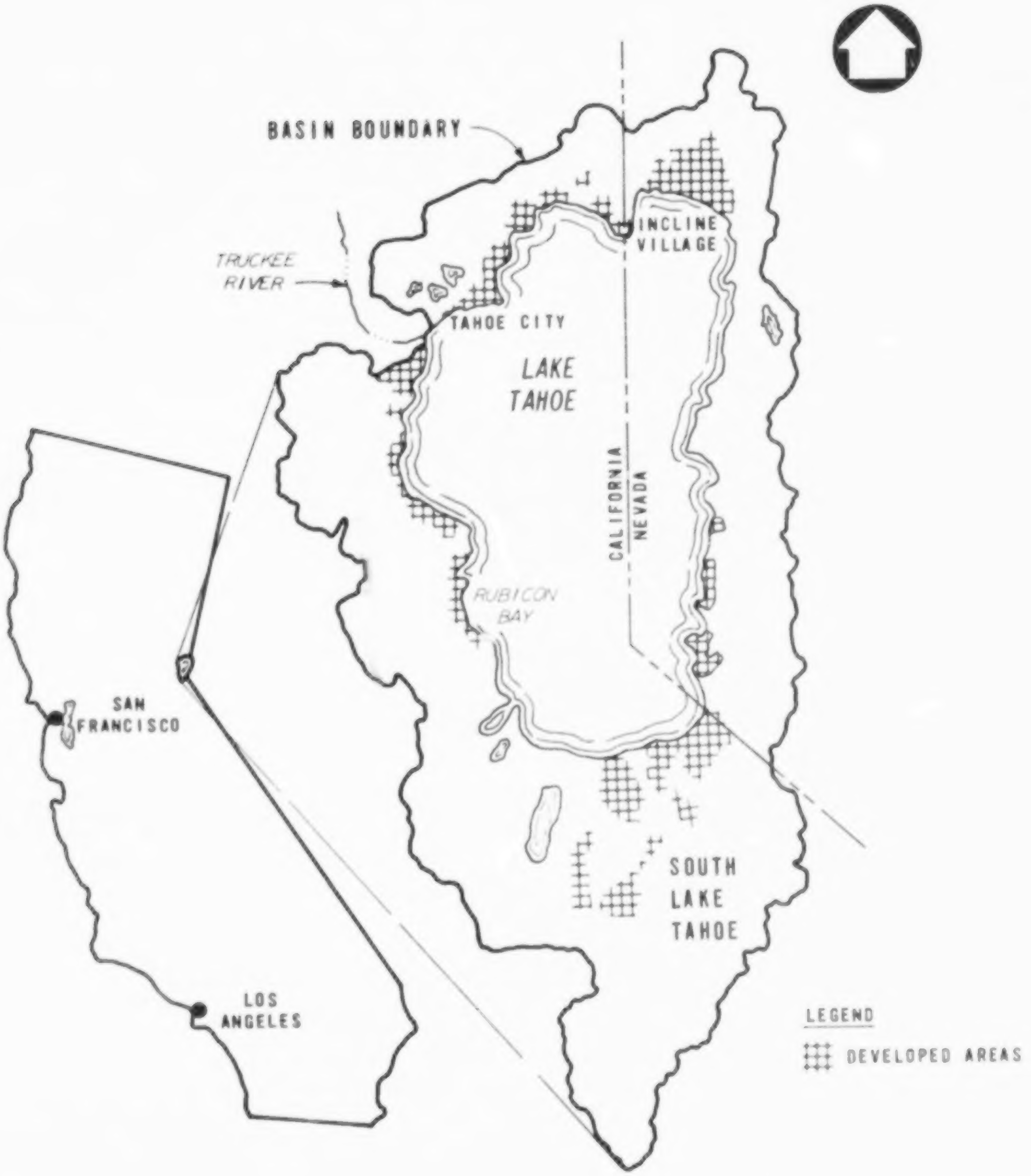


Figure 25. Lake Tahoe Basin and developed areas.

Problem Assessment

The problems caused by stormwater runoff in Lake Tahoe are the transport of nutrients and sediment to the highly pure water of the lake. Nutrient loading to the lake is considered more of a potential problem than sediment.

Nutrient Problems--

Lake Tahoe is naturally low in nutrient concentration and is considered relatively infertile. However, where most lakes respond to nutrient increases in the range of parts per million, algae in Lake Tahoe respond to concentration increases that are at least 1,000 times lower (parts per billion) [3]. Lake Tahoe is considered a nitrogen-limited lake--small increases in nitrogen can trigger algal blooms. Long-term macrobenthic studies and baseline water quality sampling indicate Lake Tahoe is of high purity; however, they also show trends toward higher rates of eutrophication as measured by increases in paraphyton levels and primary productivity. During the period 1968 through 1971, the primary productivity increased by about 25%. Compared with data from 1959-1960, the increase was over 50%, as shown in Table 43 [4, 5].

Table 43. INCREASES IN PRIMARY PRODUCTIVITY RATES IN LAKE TAHOE [4, 5]

| Year | Primary productivity, mg $C/m^2 \cdot yr$ | Percent increase over previous year, % |
|---|---|---|
| 1959 - 1960 | 38,958 | -- |
| 1968 | 46,685 | 20% since 1959 - 1960 |
| 1969 | 50,525 | 8.2 |
| 1970 | 52,467 | 3.8 |
| 1971 | 58,655 | 11.8 |

Stimulation of algal growth is evident along shore areas close to developed areas of the basin. Visible changes are apparent; green algae cling to the lake bottom and, during the spring, grow rapidly along the shore forming long hair-like strands on mooring ropes, buoys, boats, docks, and on submerged boulders [6].

Nutrient sources have been linked to stormwater and snowmelt runoff and to groundwater inflow. High levels of nitrogen have been measured in the groundwater entering the lake. The source of this nitrogen is not known; however, this loading may be caused by old leach fields used before construction of the sewerage system. One of the major sources of nitrogen has been attributed to soil disturbances from construction and development activities. Natural nitrogen fixation may also be another source of loading.

Sediment Problems--

Sediment transport to the lake also contributes to water quality degradation; however, most sediments are sand sized with little clay; and turbidity in the lake from these sediments is short lived. Sediment and particulates once in the lake provide surface attachment sites for microbial communities [7]. Problems of siltation and sediment deposition in the receiving streams draining to Lake Tahoe have occurred from erosion of unstable slopes and soil loss from developed and developing areas. Stream siltation, as shown in Figure 26, has changed the nature of many streams in the basin and has

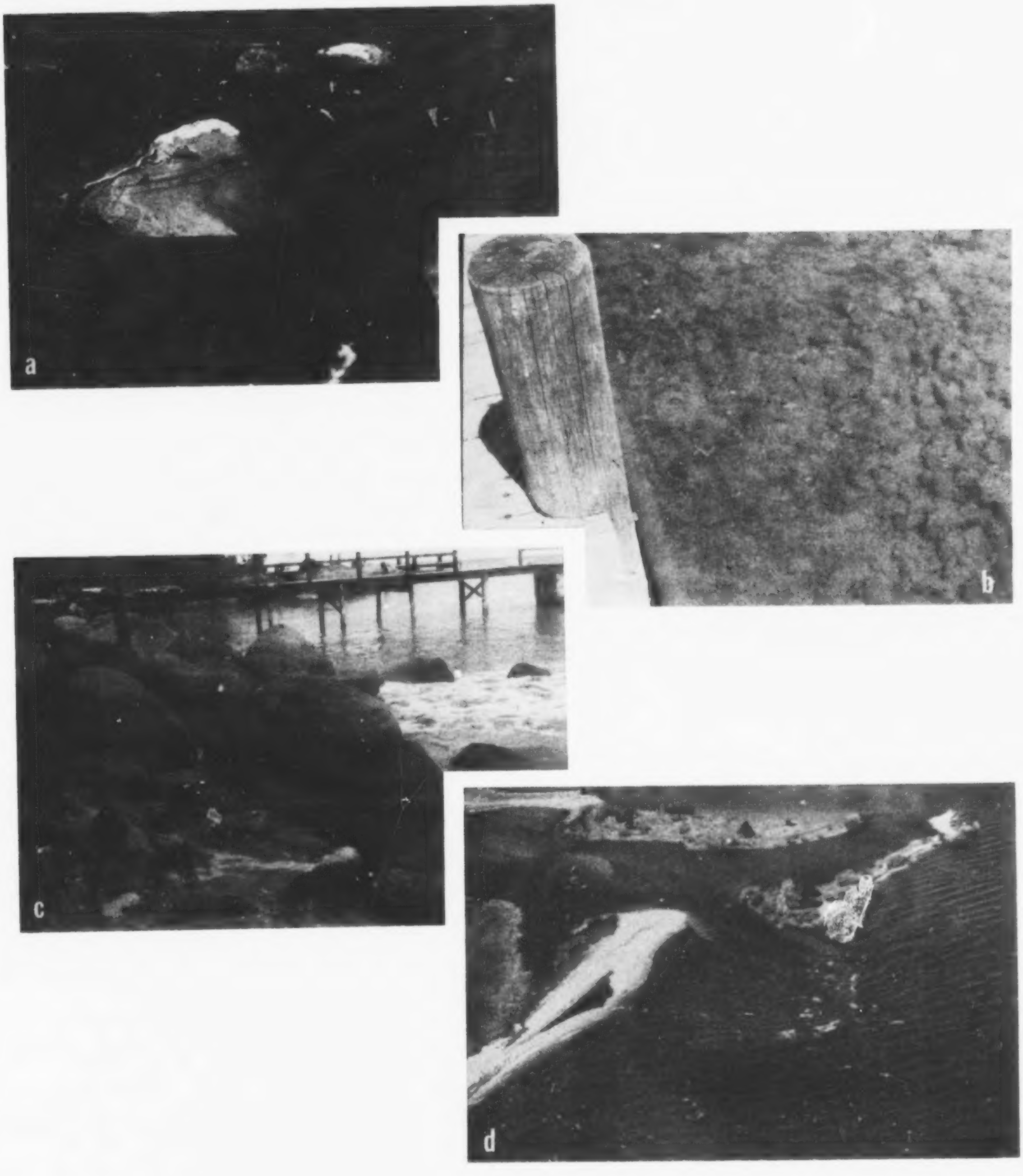


Figure 26. Stormwater problems in the Lake Tahoe Basin: (a) stream siltation, (b) algal growth in nearshore areas of the lake, (c) stormwater discharge to the lake, and (d) stormwater sediment plume.

affected the lake's appearance as well as changed the macroinvertebrate species found in the streams and the lake.

Sources of Pollutants--

Pollutant loadings to the lake come from many sources around the lake; most are a result of land activities and land disturbances caused by man. Land disturbances that result in erosion and siltation include construction activities, roadside drainage, sewering activities, unstabilized road cuts and fills, land clearance, unpaved roads and parking lots, and channelization of stream beds. Pollutant contributions from land use activities are compared in Table 44.

Table 44. COMPARISON OF MEAN RUNOFF WATER QUALITY FOR SEVERAL LAND USES AND ACTIVITIES IN THE TAHOE AREA [2]
mg/L except as noted

| Land use or activity | Suspended solids | Turbidity, Ftu | $NO_3$-N | Total nitrogen | Total phosphate | Total iron | Oil and grease |
|---|---|---|---|---|---|---|---|
| Unpaved parking lots | 16,600 | 1,000 | -- | 9.2 | 3.4 | 3.4 | 76.0 |
| Bare areas | 989 | 319 | 0.3 | 4.0 | 1.7 | 1.9 | 8.0 |
| Unsurfaced roads and driveways | 7,780 | 5,060 | 0.9 | 2.6 | 1.2 | 3.2 | 38.1 |
| Paved parking lots | 320 | 107 | 0.6 | 3.8 | 1.6 | 1.0 | 42.6 |
| Dirt roadside ditches | 648 | 175 | -- | 3.2 | 1.0 | 1.1 | 28.4 |
| Unstable dirt channels | 613 | 305 | 0.1 | 1.2 | 1.0 | 0.8 | 31.3 |
| Paved streets | 680 | 280 | 0.1 | 1.2 | 0.9 | 0.9 | 23.8 |
| Snow storage | 136 | 90 | 0.1 | 3.5 | 0.6 | 0.2 | 9.6 |
| Rooftop drainage | 30 | 7 | 0.02 | 0.8 | 0.5 | 4.7 | 7.1 |
| Roadway slopes | 443 | 304 | 0.2 | 1.0 | 0.7 | 0.5 | 6.7 |
| Construction sites | 8,630 | 764 | 0.1 | 4.0 | 0.5 | 2.3 | 0.1 |
| Corporation yards | 435 | 142 | 0.1 | 3.3 | 0.8 | 7.7 | 56.6 |
| Mobile home parks | 5,680 | 931 | 0.1 | 0.9 | 0.8 | 4.4 | 23.9 |
| Service stations | 281 | 112 | 0.2 | 0.8 | 0.9 | 1.3 | 11.7 |
| Stables | 71 | 27 | 0.02 | 1.8 | 2.2 | 6.2 | 9.1 |
| Land use types | | | | | | | |
| Tourist commercial | 4,020 | 1,084 | 0.4 | 1.3 | 0.8 | 4.2 | 67.7 |
| General commercial | 773 | 832 | 0.2 | 1.7 | 1.3 | 1.1 | 33.0 |
| Public service | 323 | 105 | 0.1 | 1.9 | 0.8 | 4.3 | 23.8 |
| High density residential | 249 | 92 | 0.1 | 0.7 | 0.8 | 1.4 | 20.0 |
| Medium density residential | 489 | 52 | 0.04 | 0.6 | 0.5 | 0.4 | 3.6 |
| Low density residential | 613 | 169 | 0.1 | 1.2 | 0.7 | 0.3 | 0.8 |
| Recreation | 48 | 21 | 0.1 | 0.6 | 0.4 | 0.5 | 5.3 |
| General forest | 66 | 6 | 0.03 | 0.2 | 0.1 | 0.4 | 0.6 |
| General urbanized area | 482 | 242 | 0.1 | 1.1 | 0.8 | 1.3 | 34.4 |

Pollutant problems in the lake have not yet reached major proportions, but do indicate the need for preventive programs to arrest these problems before irreversible damage and impacts result.

### Countermeasure Philosophy (208 Planning)

A basinwide countermeasure approach has been developed for water quality problems through 208 planning. A portion of the plan addresses land use planning and erosion control measures for limiting stormwater runoff. These BMPs, enforced by ordinances and regulations, are applicable to developing and developed areas of the basin.

The 208 planning goals for these practices include expected reductions of 80% for suspended solids, 40% for nitrate-nitrogen, 70% for total nitrogen, and 80% for total phosphorus. Overall implementation costs to control stormwater runoff in the Lake Tahoe Basin are expected to reach about $97 million (ENR 3000) [1].

#### Developing Areas--

By controlling development through land use planning, land use capability features can be identified and restrictions of use imposed to prevent runoff problems. Evaluation of land use restrictions is based on land use capability: the ability of the land to withstand disturbance caused by development.

Much of the Tahoe area is considered fragile and consists of steep slopes, poorly drained soils, and areas of sensitive or relatively spare vegetative cover. Three levels of land capability have been identified according to risk or potential land damage or disturbance [3]:

1. Lands that should remain in their natural condition
2. Lands that can permit certain uses
3. Lands that are most tolerant to urban uses

#### Developed Areas--

Erosion countermeasures are used to control stormwater runoff problems in existing developments if proper planning and management techniques were not used. In developing areas, they can solve specific problems that are unavoidable or that cannot be controlled by land use planning programs. Erosion controls include temporary soil stabilization, slope stabilization, temporary runoff management, runoff collection and conveyance structures, control of runoff from impervious surfaces, and revegetation of disturbed areas [8].

Soil stabilization techniques are only temporary and should be used as control measures during construction or until permanent stabilization, such as vegetation, is established. Control measures include hydromulching or application of wood chips or straw, with or without tackifiers, and netting or matting to prevent soil loss and to provide a medium for vegetative growth.

Slope stabilization involves reshaping erosion hazard slopes and can include retaining walls, benches, or serrations to prevent erosion and soil loss.

Examples of temporary runoff management measures are sandbag or straw bale sediment barriers, filter berms and fences, and filter inlets. These methods are particularly applicable to construction activities where runoff from construction sites is to be controlled. Permanent measures to control runoff from slopes include diversion dikes, subsurface drainage, runoff interception trenches, chutes, or flumes, and level spreader areas.

For runoff collection and conveyance, available controls are catchbasins, curbs and gutters, roadside ditches, storm drains, check dams, and source detention facilities.

Use of pavements and proper design of parking lots, service aprons, driveways, and corporation yards provide control for runoff from impervious surfaces. Use of porous pavements and dripline trenches can also be considered.

To revegetate disturbed areas, the following can be considered: selection of vegetation types and planting techniques, seedbed preparation, maintenance, and fertilizer use. Use of soil and slope stabilization measures or other BMPs previously addressed are interrelated and may be used to assist revegetation efforts.

The 208 study has identified potential erosion and drainage hazard areas in the Tahoe Basin, as shown in Table 45, where source control measures could be used.

Table 45. SUMMARY OF EROSION AND DRAINAGE PROBLEM AREAS IN THE TAHOE BASIN [1]

| Description | Erosion hazard rating | | | Basinwide total |
|---|---|---|---|---|
| | High | Moderate | Slight | |
| Unvegetated roadway slopes, acres | 201 | 84 | 169 | 454 |
| Over-steepened roadway slopes, mi | 51 | 18 | 15 | 84 |
| Areas stripped of vegetation, acres | 114 | 32 | 75 | 221 |
| Eroding roadway shoulders, mi | 68 | 101 | 293 | 462 |
| Unstable drainage systems, mi | 21 | 30 | 67 | 118 |
| Eroding dirt roads, acres | 63 | 61 | 53 | 177 |

### Legislation--

The use of land use planning criteria and erosion controls in the Lake Tahoe Basin depends on the adoption and enforcement of ordinances and regulatory programs. The regional planning agency has developed a number of ordinances to enforce compliance with planning measures to control stormwater runoff pollution [9], including:

- Land use ordinance - establishes land use districts, limits density and land coverage, and provides procedural requirements for land use matters.

- Grading ordinance - regulates cuts, clearing of vegetation and construction and maintenance of landfills, and sets revegetation standards.

- Subdivision ordinance - regulates the subdivision of land and establishes procedures required for such subdivision.

- Shoreline ordinance - establishes standards and regulates shoreline development, filling and dredging, and the construction, alteration, removal, and maintenance of shoreline structures.

Land use controls will be regulated through review and permit procedures for new developments. For developed areas, compliance with water quality goals may have to be enforced through legislative action. Economic and social impacts of the latter, however, may adversely affect individual landowners.

## ASSESSMENT OF LAND USE PLANNING

Land use planning, source control concepts that can prevent and reduce sources of stormwater pollution, is a most promising countermeasure approach. The goal of planning is to preserve the natural ecological balance of an area in terms of volume, runoff rate, and pollutant characteristics by recognizing sensitive areas and restricting development in those areas.

An assessment and comparison of land use planning to show methods of planning, effectiveness, impacts, and potential costs is made between two developments in the Tahoe area, Site 1 and Site 2. Site 1 is a well planned and constructed residential/recreational development built in the early 1970s. Site 2 is a residential subdivision constructed in the late 1950s and early 1960s and lacks many of the planning and construction controls necessary for environmental protection.

### Approach to Land Use Planning

Site 1 was planned as an all-year recreational and resort community and includes both a major ski complex and golf course. Planning activities were guided, recognizing the qualities and limitations of the surrounding landscape and environment to develop a harmonious combination of land use activity and environmental protection. Erosion control considerations of land use planning included (1) restricting land uses to suitable sites that could support the intended activity, (2) minimizing land disturbances (road cuts and fills), and (3) protecting stream environment zones.

#### Land Use Planning Elements--

The planning activities at Site 1 centered on a multidisciplined approach involving an iterative evaluation of the following elements:

- Physical analysis of the site - identification of the environment--soils, geology, slopes, drainage, access, and types of development suitable for the site.

- Market analysis - identification of public needs and interests in the types of activities and facilities to be developed.

- Economic analysis - examination of costs and profitability to develop facilities within the limits of the land available for development.

- Regulatory requirements - coordination with local, regional, and state agencies to conform with environmental legislation and ordinances.

#### Implementation of Land Use Planning--

Implementation of land use planning at Site 1 involved identification and mapping of the physical features of the site and screening of sensitive and environmental hazard areas by applying land use criteria. Identification of developable areas and development types having minimum conflicts with the physical features of the area resulted after screening the areas with sensitive vegetation types, slopes too steep for development, areas within stream zones and flood plains, and areas of poorly drained soils. The most important screening considerations were slope and drainage, with vegetation type and density, soils, and exposure and snow depth being secondary. Approximately 70% of the open space areas was to remain undeveloped. The remaining 30% was planned for residential, commercial, utilities, roads and parking, and recreational facilities.

Site 2, on the other hand, represents an uncontrolled development with little apparent concern or controls placed on sensitive physical features of the site. This development had no provision for open space or for limiting encroachment of development in stream zones and drainage areas. Roads and residential housing were constructed without regard to slope or erosion potential and, therefore, required extensive erosion controls to protect land features and receiving water quality.

### Description of the Project Sites

Both project sites are on mountainous terrain in the Tahoe area. Site 2 is in the Lake Tahoe Basin, and Site 1 is just north of the basin, as shown in Figure 27.

The characteristics of each site, including type and method of development, highlight the difference between a high degree of environmental planning and no planning at all. The effects are reflected in the postdevelopment water quality conditions and sediment yields from each site.

#### Well-Planned Development--

The development at Site 1 consists of 1,036 ha (2,560 acres) of a privately owned, 10,500 ha (25,900 acre) tract north of Lake Tahoe. In addition to a planned ultimate residential development of 585 single-family lots and 3,115 condominium units, the area includes a 68 ha (168 acre) golf course and a 132 ha (325 acre) ski area. A summary of land use areas at Site 1 is shown in Table 46 [10].

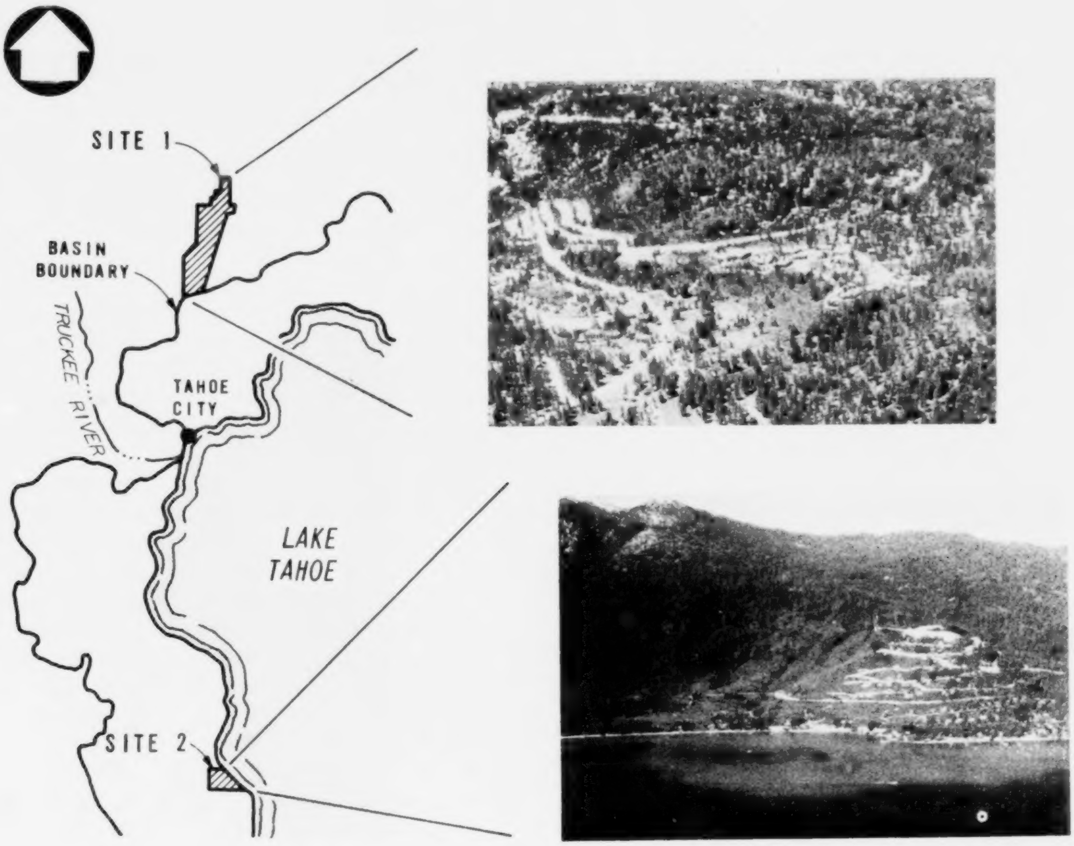


Figure 27. Location of the well planned and poorly planned project sites in the Tahoe area.

Table 46. SUMMARY OF PLANNED LAND USE AREAS AT SITE 1 [10]

| Land use | Area, acres | Percent of total area |
|---|---|---|
| Open space | 1,825 | 71.3 |
| Developed area | | |
| Residential | 88 | 3.4 |
| Commercial | 17 | 0.7 |
| Utilities | 20 | 0.8 |
| Roads and parking | 118 | 4.6 |
| Recreational facilities | 491 | 19.2 |
| Subtotal | 734 | 28.7 |
| Total area | 2,559 | 100.0 |

*Land Use Planning and Erosion Controls*--The development of the land use types at Site 1 included preplanned erosion controls and provision for compatible construction with the environment. Erosion potential from the ski areas was reduced by proper site selection of the ski runs and facilities. In most cases, ski runs were cut on less than 3:1 slopes, diagonal to the fall line, to route runoff to undisturbed areas adjacent to the runs. The use of flow barriers on steeper slopes and maintaining natural vegetation or implementing comprehensive revegetation programs prevented erosion in cleared and disturbed areas.

Cuts and fills for street and parking lot construction were limited to slopes no steeper than 1-1/2:1 for cuts and 2:1 for fills. To reestablish native vegetation, topsoil was stockpiled and replaced after construction. Propagation of native vegetation and seeds by this method proved successful in most areas where slopes were not too steep.

Planned construction of residential and commercial buildings was limited to slopes of less than 15%. This restricted the developable area of the site to relatively flat ground on ridgelines and in valley floors. The area conforming to this criteria was limited, however, and actual construction extended beyond these planned limits. In these areas, erosion control practices were implemented to correct potential problems. In all cases, no development was constructed on slopes steeper than 25%, with most less than 20%. Other practices during construction included: (1) minimizing surface disturbances, (2) using check dams and erosion baffles, (3) lining drainage channels, (4) using filter fences and berms to prevent siltation from contruction site erosion, and (5) using slope stabilization techniques. These and other erosion control measures are covered later in this case study.

The development at Site 1 was constructed to blend in with the natural surroundings and created minimal environmental disturbances, as shown in Figure 28.

*Water Quality*--The surface water quality leaving the development at Site 1 showed slight increases in suspended sediment concentrations above estimated background levels. Suspended sediment is the heavy solids that cause sediment deposition or siltation in the stream as the flow recedes. The suspended sediment concentration is proportional to three types of runoff conditions and runoff rates: low or base flow, rainfall, and snowmelt.

Rainfall causes the highest concentration of suspended sediment with the highest monitored values from the entire site in the range of 500 to 600 mg/L. The average concentration leaving the site during rainfall is 115 mg/L. The average concentration for low flow is 7.9 mg/L and for snowmelt conditions is 28.5 mg/L [10].

Suspended sediment loads from the site represent approximately a 100% increase over predevelopment conditions, 12 T/km$^2$·yr (107 lb/acre·yr) to 24 T/km$^2$·yr (214 lb/acre·yr).

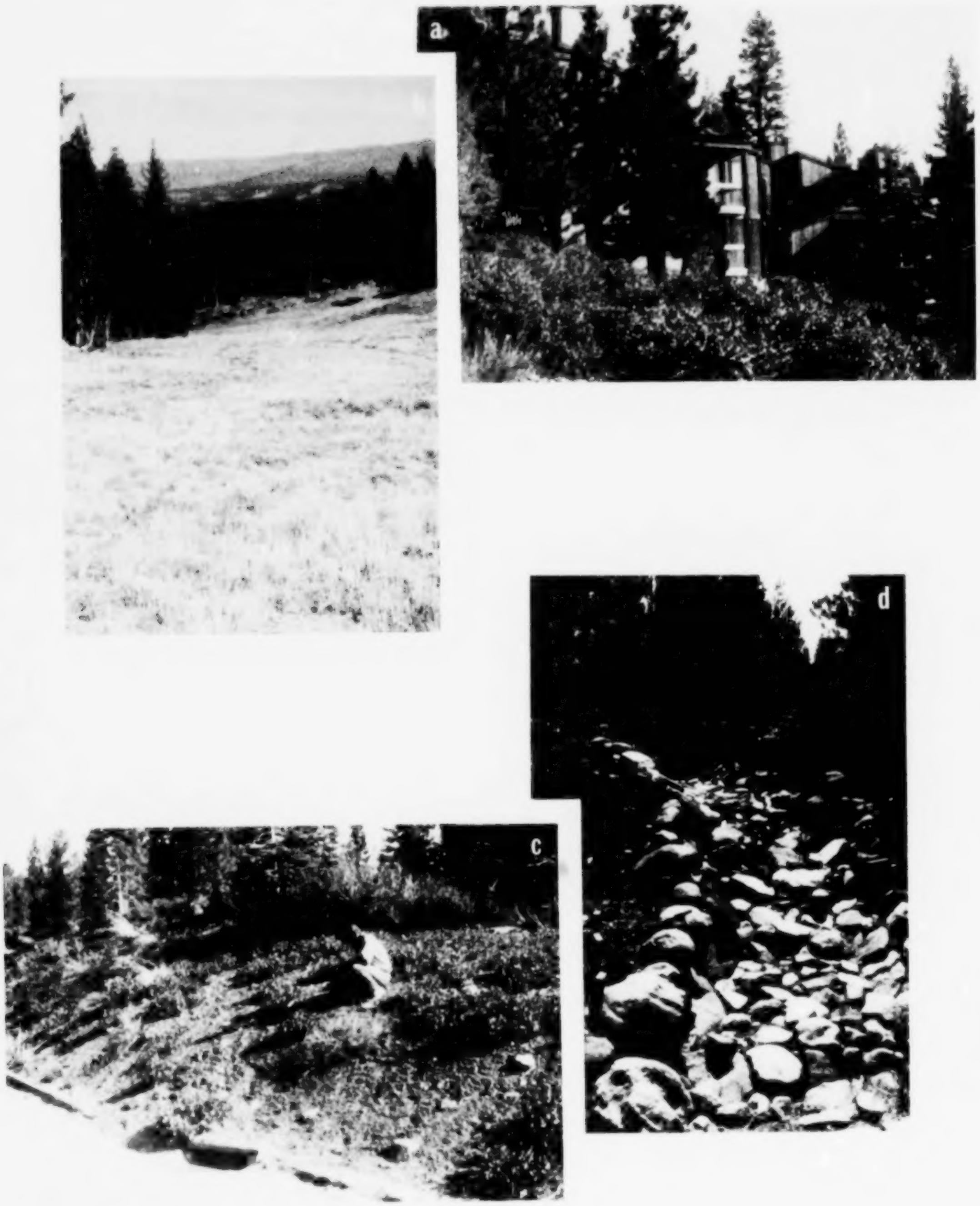


Figure 28. Results of land use planning at Site 1: (a) condominimum development and undisturbed natural surrounding, (b) revegetated ski slope, (c) revegetated gentle-sloped road cut, and (d) rock-lined natural drainage channel.

### Poorly Planned Development--

The development at Site 2 is a single-family residential subdivision of 128 ha (316 acres), with 632 subdivided parcels. The upper portions of the site are on steep slopes and have a high potential for erosion. A 24 ha (60 acre) portion of this upper area was selected as a demonstration site for implementation of erosion controls to correct the effects of poor initial planning [10]. The site is shown in Figure 29.

The upper portions of this site have been under continuous development without land use restriction controls. Currently, only 19% of the 24 ha (60 acre) site has been developed. Virtually all of the site area is on steep slopes ranging between 30 and 60%, with poor soil drainage characteristics. Regional planning guidelines rate the site in the lowest land capability class and allow only a 1% impervious surface coverage [10]. At full development of the site, it has been estimated that half of the area would be impervious, coverage, and disturbed or unvegetated slopes. A summary of the land uses and impervious areas before implementation of the erosion control project is shown in Table 47.

Table 47. SUMMARY OF LAND USES AND IMPERVIOUS AREAS AT SITE 2 BEFORE EROSION CONTROL PROJECT [10]

| Description | Area, acres | Percent of total area |
|---|---|---|
| Impervious coverage on private lots | 2.8 | 5 |
| Road surfaces | 6.7 | 11 |
| Disturbed and unvegetated slopes | 8.1 | 13 |
| Undisturbed area[a] | 42.5 | 71 |
| Total area | 60.1 | 100 |

a. Site 2 has no provision for open space.

Construction of homes and roads is continuing on the steep slopes of the site. Embankments that were originally cut at 1:1 slopes or greater have eroded to 1-1/2:1 or less. Several roads on the site have grades steeper than 15% and present both maintenance problems during snow removal and traffic hazards.

The steep slopes and development within drainage areas and in stream zones add to the stormwater drainage problems. Drainage ditches, curbs, gutters, and culverts at the site are generally undersized for the high flows that occur. These facilities are constantly clogged with eroded soil from the disturbed areas. The subdivided area at Site 2 has no open space areas and has lots adjacent to streambanks and in drainage areas. The development and examples of poor land use planning are shown in Figure 30. Development in stream or drainage zones allows runoff to enter receiving waters directly from the disturbed areas.

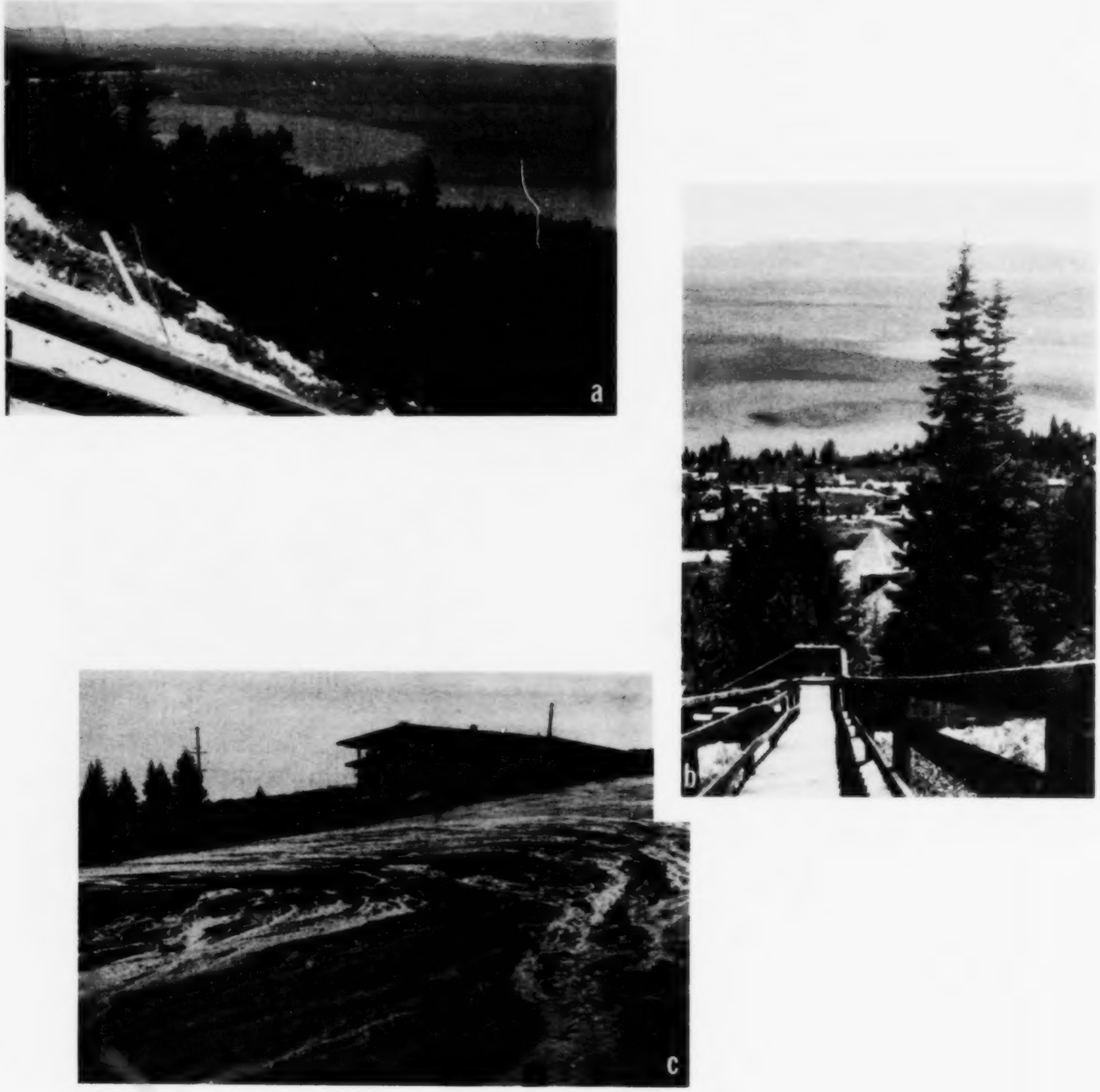


Figure 29. Site 2 at Lake Tahoe: (a) and (b) development in the upper portion of the site on extremely steep slopes, and (c) resulting erosion problem from road cut.

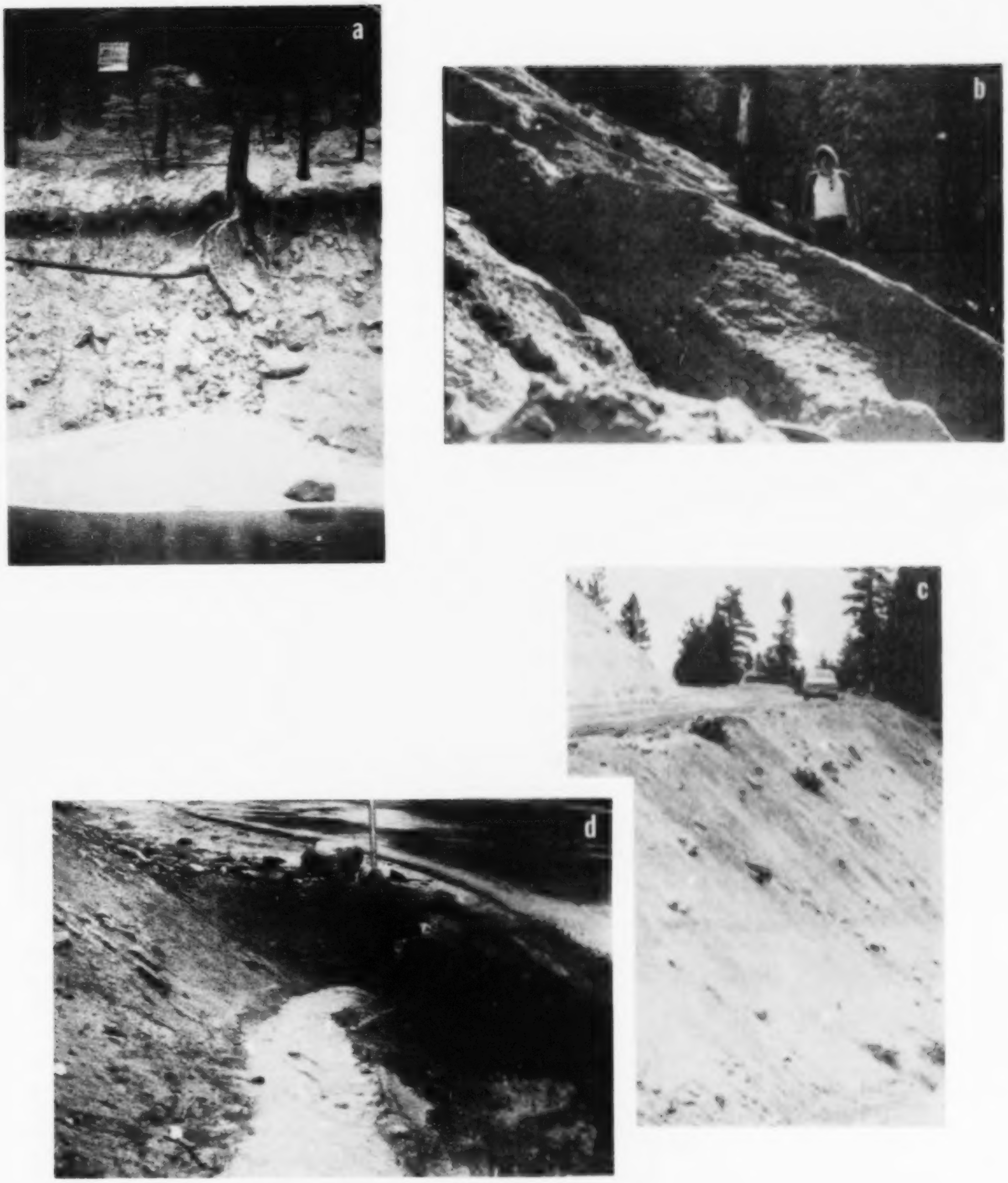


Figure 30. Results of uncontrolled development at Site 2: (a) unstable road cut, (b) eroded road fill, (c) steep road grades, and (d) sediment-laden stormwater entering storm culvert.

Water quality problems from Site 2 are related to suspended sediments carried to the receiving waters by stormwater runoff. Estimates of erosion rates from unstable cut and fill slopes averaged 1,975 $T/km^2 \cdot yr$ (17,600 lb/acre·yr). Unit sediment yields from the development represented more than a 100-fold increase over measured background levels: from 3.4 $T/km^2 \cdot yr$ (30 lb/acre·yr) to 366 $T/km^2 \cdot yr$ (3,260 lb/acre·yr).

Rainfall produces the highest suspended sediment concentrations from the site. The highest monitored concentration was measured at over 15,000 mg/L, with the average at about 1,800 mg/L. A comparison of the average suspended sediment concentrations for various flow conditions entering and leaving the developed area is shown in Table 48.

Table 48. AVERAGE INSTREAM SUSPENDED SEDIMENT CONCENTRATIONS ABOVE AND BELOW SITE 2 [10]
mg/L

| Flow condition | Above development[a] | Below development |
|---|---|---|
| Low flow | 1.3 | 12.9 |
| Rainfall | 20.8 | 1,798.1 |
| Snowmelt | 9.1 | 434.6 |

a. Represents background levels.

## Economic and Environmental Impacts of Land Use Planning

The costs of land use planning for environmental protection are difficult to quantify. Most costs are not directly related to a specific control measure but rather to an overall development philosophy that includes aesthetics and profitability, in addition to meeting regulatory requirements. Environmental impacts are characterized by the quality of the receiving waters draining the controlled and the uncontrolled development.

### Costs of Land Use Planning--

Two cost components were identified to assess the cost of land use planning at Site 1. These include predevelopment planning costs, \$443,000, and costs to construct erosion control measures where needed, \$127,000. Assuming that all predevelopment planning costs and the required erosion controls are attributed to environmental protection, the total control costs representing land use planning for Site 1 are \$570,000 (ENR 3000), or \$550/ha (\$220/acre) of total development area [10].

The average projected cost per condominium unit, at full development, is approximately \$360. This cost could be passed on to the purchaser to cover the cost of planning and preplanned erosion control measures.

Environmental Impacts of Land Use Planning--

The effectiveness of land use planning is shown by a comparison of the sediment yields of the developed areas at Site 1 and Site 2. Although development slightly increases sediment yields even in well-planned developments, the disregard of land use planning criteria to protect sensitive areas can produce the extreme results shown in Figure 31.

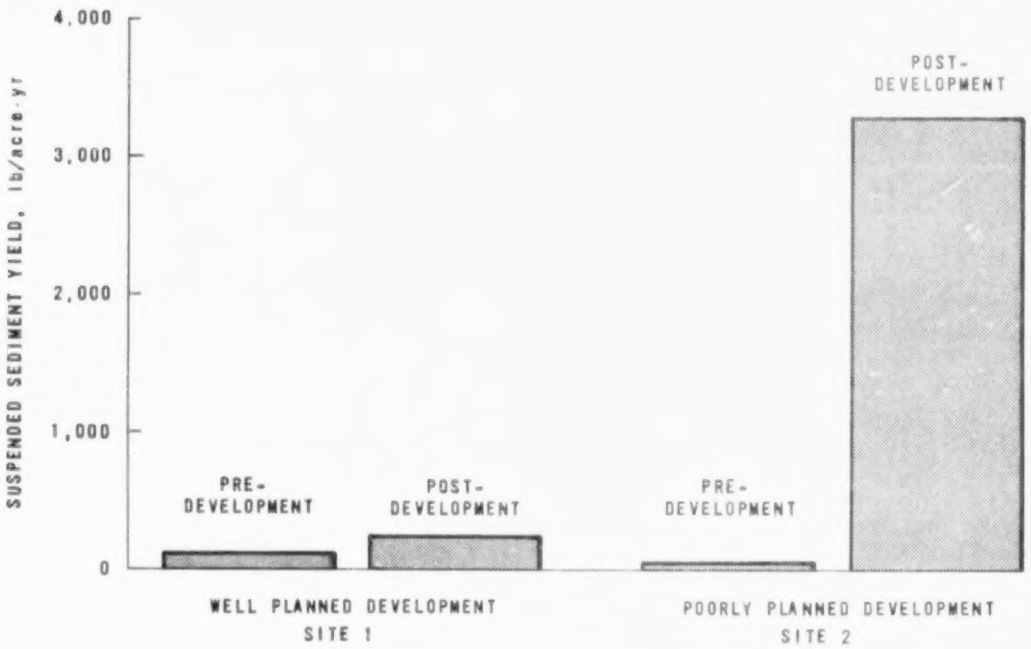


Figure 31. Comparison of sediment yields from a well planned and a poorly planned development [10].

Environmental impacts can be characterized by the interrelationship of the receiving water quality of the streams (sediment deposition) and the health of the stream and macrobenthic communities.

Impacts of suspended sediment on the macrobenthic community increase as sediment loadings increase. The changes in the macrobenthic community, at sampling stations above and below the poorly planned development at Site 2, are shown in Table 49. These changes result from about a 100-fold increase in sediment load over background conditions.

Although a twofold increase in suspended sediment load was experienced at Site 1, the impact of this increase on the macrobenthic community was negligible. Several monitoring stations near areas of soil disturbance showed some fluctuations in the number species, density, and diversity of species. However, for the development as a whole, monitoring downstream of the development indicated only minor impacts compared with upstream monitoring, as shown in Table 50.

Table 49. MACROINVERTEBRATE SAMPLING RESULTS ABOVE AND BELOW THE POORLY PLANNED DEVELOPMENT SITE 2 [10]

| Date of sampling | Density, No./$m^2$ | | Number of species | | Species diversity index[a] | |
|---|---|---|---|---|---|---|
| | Above | Below | Above | Below | Above | Below |
| Jul 1975 | 1,542 | 267 | 20 | 9 | 2.50 | 1.91 |
| Dec 1975 | 1,321 | 277 | 19 | 12 | 2.25 | 2.08 |
| Jun 1976 | 2,125 | 1,652 | 14 | 14 | 2.15 | 1.85 |
| Oct 1976 | 1,560 | 19 | 14 | 4 | 2.21 | 1.35 |

a. A measure of the relationship between the number of species and the total biologic community population by the Shannon-Weaver index: diversity = $-\Sigma(Ni/N) \ln (Ni/N)$, where $Ni$ = number of species and $N$ = total community population.

Table 50. MACROINVERTEBRATE SAMPLING RESULTS ABOVE AND BELOW THE WELL-PLANNED DEVELOPMENT SITE 1 [10]

| Date of sampling | Density, No./$m^2$ | | Number of species | | Species diversity index | |
|---|---|---|---|---|---|---|
| | Above | Below | Above | Below | Above | Below |
| Sep 1974 | 619 | 2,193 | 19 | 20 | 2.78 | 2.26 |
| Jul 1975 | 785 | 1,611 | 20 | 22 | 2.01 | 2.06 |
| Dec 1975 | 1,002 | 821 | 19 | 20 | 1.69 | 2.29 |
| Jun 1976 | 995 | 1,364 | 18 | 14 | 1.69 | 1.55 |
| Oct 1976 | 1,212 | 1,578 | 15 | 20 | 2.44 | 2.08 |

## EROSION CONTROLS

Erosion controls mitigate erosion and sediment problems where land use activities have disturbed soil surfaces or created erosion hazard areas. Even in the well-planned development, erosion controls were used; however, they were implemented according to a planned strategy where land disturbances were unavoidable. For developments with no land use planning, such as at Site 2, erosion controls serve as the only mitigation option available to correct sediment and erosion problems.

The erosion controls at Site 2 were implemented as part of a demonstration project to determine the effectiveness of a large number of different control technologies and variations of similar technologies. Side-by-side plots were used on most control areas where different controls were applied for comparison [10].

Erosion controls can be temporary measures that are effective for short periods until permanent controls can be established or until land disturbance is minimized after construction. Permanent controls are long-term countermeasures used to control erosion from disturbed areas, such as road cuts and fills, that are physically unstable or exposed to rain or runoff over long periods. These controls usually employ mechanical stabilization methods.

### Description of Demonstration Project

Approximately 2.9 ha (7 acres) of disturbed area was identified in the 24 ha (60 acre) area of Site 2 for implementation of erosion controls. These disturbed areas, for the most part, represent road cuts and fills, as shown in Figure 32. The principal control measures were permanent and temporary slope stabilization controls and revegetation. Other control measures were temporary runoff and siltation control, runoff control on slopes, and conveyance systems. Over 200 separate plots were used to demonstrate various erosion control measures within the site [10].

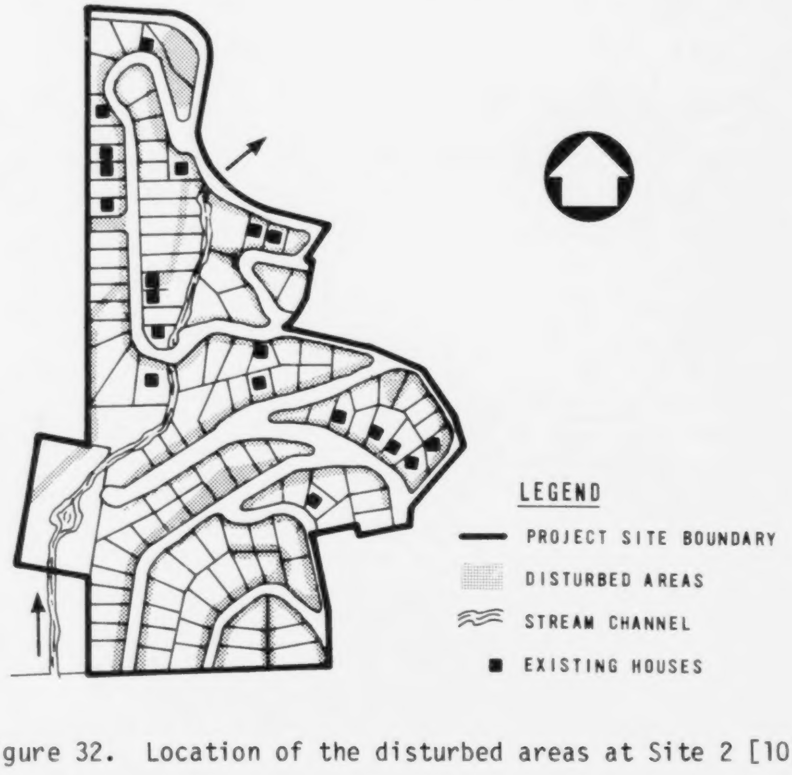


Figure 32. Location of the disturbed areas at Site 2 [10].

### Temporary Soil Stabilization--

Temporary soil stabilization methods are used on disturbed slopes or on areas to provide erosion control, dust control, mulch or mulch protection, or a medium for applying or holding seeds during revegetation. The effectiveness of these methods is short lived. Sediment and nutrient yield control, compared to bare ground, is most effective for 6 months to 1 year, with a steady decrease up to 2 years, and a sharp drop in effectiveness after 2 years. A description of temporary soil stabilization methods and their projected effectiveness is presented in Table 51.

Table 51. DESCRIPTION AND EFFECTIVENESS OF TEMPORARY SOIL STABILIZATION METHODS [8]

| | | Effectiveness, %[a] | | | |
|---|---|---|---|---|---|
| | | Sediment | | Nutrients | |
| Method | Description | Initial | 2 yr | Initial | 2 yr |
| Jute matting | Mulch nets made of jute used for erosion control and protection of other mulches. | 70-90 | 40-60 | 50-70 | 20-50 |
| Matting in drainage channels | Application of jute matting or fiberglass roving for dust and erosion control in very small drainage channels with flow velocities less than 2 ft/s. | 50-90 | 20-60 | 30-70 | 10-50 |
| Plastic netting | Monolithic plastic clothlike material used over mulch, straw, or hydromulch. | Provides no control by itself | | | |
| Wood excelsior matting | Mat of wood excelsior fibers bonded to a paper or plastic used for dust and erosion control. Flows under mat should be prevented. | 50-90 | 20-60 | 30-70 | 10-50 |
| Fiberglass roving | Matting of continuous strands of glass fibers and tacking agent. Used for dust and erosion control and as a mulch for seeded and unseeded areas. | 90-95 | 80-90 | 60-80 | 50-70 |
| Hydromulching | Mechanized rapid method for applying wood fiber mulch, and tacking agent with or without seeds to large areas. | 70-90 | 40-60 | 50-70 | 20-50 |
| Wood chip application | Temporary mulch and surface protection using chips of wood. Used for dust and erosion control during construction and as a mulch around plantings. | 90-95 | 80-90 | 60-80 | 50-70 |
| Crushed gravel mulch | Application of gravel or crushed stone as a mulch to stabilize soils during construction, or for low-use dirt roads, driveways, and areas of light vehicular use. | 70-90 | 70-90 | 50-70 | 50-70 |
| Straw mulch | Application of staple straw as a protective cover over bare or seeded soil to reduce erosion and provide a mulch. Requires matting or other methods to hold it in place. | 90-95 | 40-60 | 60-80 | 20-50 |

a. Effectiveness for most methods after 2 years usually ranges between 0 and 10%.

### Permanent Slope Stabilization--

Permanent slope stabilization controls are mechanical methods used to physically change the disturbed slope area or provide physical barriers to support the slope. The following methods, described in Table 52, do not provide for mulch or surface protection to bare slopes and require temporary slope stabilization methods until permanent vegetation is established. In many of the corrective measures implemented at Site 2, one or more of these methods were used to provide adequate erosion protection.

Table 52. SUMMARY OF PERMANENT SLOPE STABILIZATION MEASURES [8]

| Methods | Description | Applicability |
|---|---|---|
| Rock retaining wall | A low gravity wall constructed of rock materials to provide an aesthetically attractive method for physically stabilizing a slope. | For use on slopes which are steeper than 2:1 and cannot be regraded to achieve this gradient. |
| Redwood retaining wall | A retaining wall constructed of redwood planking and posts to stabilize oversteepened or unstable slopes. | Used on small slopes of loose material underlain by rigid rock or firm subsoil to securely anchor the wall. |
| Gabions | Large, single- or multi-celled, rectangular wire mesh boxes filled with rock and wired together for permanent slope or drainage stabilization and erosion control. | Used as retaining walls to stabilize oversteepened slopes, or slope facing (revetments) [illegible], channel linings, culvert headwalls and aprons particularly where seepage is anticipated. |
| Slope bottom bench | A gently sloping surface at the base of a steeper slope to retain eroded material. | Used to control erosion on small oversteepened slopes (20 ft or less) that cannot be regraded because of access and or newly constructed slopes. |
| Wattling | Bundles of live cuttings from willows to stabilize slopes and provide revegetation. Wattling reduces slope lengths for surface runoff, increases water retention, and provides additional organic matter. | Used on slopes no steeper than 2:1 with long slope lengths providing uninterrupted paths for surface runoff. Best applied to moist sites and should not be a substitute for retaining walls or mechanical stabilization methods. |
| Slope stepping | Continuous series of horizontal steps cut on the face of cut slopes to interrupt slope length and provide slope stabilization. | Used in new construction on large cut slopes in soft rock which can be excavated by ripping. |
| Slope serration | Construction of approximately 10 in. horizontal steps on the entire face of a cut slope to provide stabilization benches that can support vegetation. | Serration is limited to slopes in medium to highly cohesive soils or in soft rock with a gentle slope (2:1 or less). Not applicable on deposited soils or in moraines. |

### Runoff Control and Temporary Runoff Management--

Runoff control and temporary runoff management measures should be considered in construction areas or used in addition to slope stabilization controls for increased effectiveness. Several of the following measures were used during construction at Site 1 and were used during implementation of the erosion control measures at Site 2:

- Diversion dike (at the top of cut or fill slopes) - diverts flows from the slopes into stable areas
- Runoff interception trench - intercepts long slope faces on gentle slopes (less than 3:1) and allows diversion and infiltration of the runoff and retention of sediments
- Pipe drops, chutes, or flumes - conducts flows down unstable slopes
- Straw bale sediment barrier - allows flows to be filtered through barrier and retains the sediment

- Sand bag sediment barrier - diverts flows and retains sediments, but does not allow water to filter through the barrier

- Gravel filter berm - removes sediments from stormwater runoff

- Filter fence (a barrier of filter cloth) - provides sediment removal for water discharged from construction sites

- Filter inlet (a temporary berm) - protects a stormwater inlet that retains sediment and allows flow to pass through

- Siltation berm (a temporary impermeable berm) - retains runoff from construction sites

### Revegetation--

Vegetation provides the best long-term erosion protection on sloped surfaces, and is the ultimate goal when applying corrective measures to disturbed areas. However, revegetation of disturbed or bare slopes by itself will not stabilize over-steepened slopes; therefore, temporary or permanent slope stabilization must be used before establishing vegetative cover. In many instances, revegetation methods can be combined into single operations with temporary stabilization methods.

The selection and use of native seeds and plantings for revegetation is a critical factor in the success and effectiveness in establishing plant growth, considering the adaptability of native vegetation to climate, soil condition, and soil type. Success of establishing new growth is also affected by slope and aspect of the disturbed area, with better growth on less steep slopes and protected exposures.

Hydraulic application of seeds to the seedbed (hydroseeding) can often be combined with the hydromulching step used for temporary slope protection. Hydroseeding applies the seed and fertilizer in a water slurry, but requires close vehicular access to the seeding area. Application of fertilizers requires control to prevent improper or excessive use to protect water quality and should only be used where soil nutrient deficiencies exist.

## Evaluation of Erosion Controls

The use of straw mulch with a chemical or mechanical tackifier is one of the most effective erosion control methods at Site 2. Using a straw mulch is as effective or better than other techniques, some of which are more expensive. Contour wattling is also an effective method to mechanically stabilize and revegetate over-steepened slopes. Growth of other types of seeded or planted vegetation is higher on slopes that received contour wattling.

Implementation of corrective erosion controls is extremely labor intensive, particularly for slope reshaping and mechanical stabilization methods.

Several maintenance practices at Site 2 have also increased the sediment load from the site or have increased the development erosion problems [10]. These include:

- Removing accumulated sediments from the toe of an eroding slope that undercuts the stability of the slope and leads to an increased erosion rate.
- Washing culverts and drains clogged by eroded sediments increases the rate of downslope sediment transport. This practice, however, does reduce upstream flooding.
- Improper disposal of waste earthen material, such as "over-the-bank" practices, increases sediment transport and hinders the proper establishment of slope stabilization measures.
- Negligence in providing revegetation or other stabilization to areas disturbed for the connection of sewer and water laterals or other underground utilities. Frequently, the disturbed surface acts as a channel for upslope storm runoff or snowmelt runoff.

These erosion control measure applications, either singularly or in combination, are shown in Figures 33 and 34.

## Unit Costs of Erosion Controls

The erosion controls at Site 2 involved a combination of many unit control measures to provide adequate slope protection, with an estimated average cost of $93,400/ha ($37,800/acre) of disturbed area. The costs ranged from $3,700/ha ($1,500/acre) for simple seeding and mulch applications to over $249,000/ha ($100,800/acre) for extensive controls, including retaining walls, wattling, plantings, seeding, and mulch applications. Overall costs to individual landowners to provide these controls could be in excess of $2,500 per lot [10].

Estimated unit costs for permanent and temporary slope stabilization methods are summarized in Tables 53 and 54, and include materials, labor, and equipment cost components.

Estimated unit costs for runoff control and temporary runoff management and other controls are summarized in Table 55.

The costs to revegetate disturbed areas can vary significantly depending on the slope, the need for slope stabilization, reshaping, or seed bed preparation, and the method of application of seed or plantings. The costs vary from approximately $2,500/ha ($1,000/acre) for seed application (hydroseeding with mulch) to over $67,000/ha ($27,000/acre) for plantings of routed shrub cuttings. Seed and fertilizer application adds about $500/ha ($200/acre) to the cost of hydromulching. Estimated costs of various revegetation methods are summarized in Table 56.

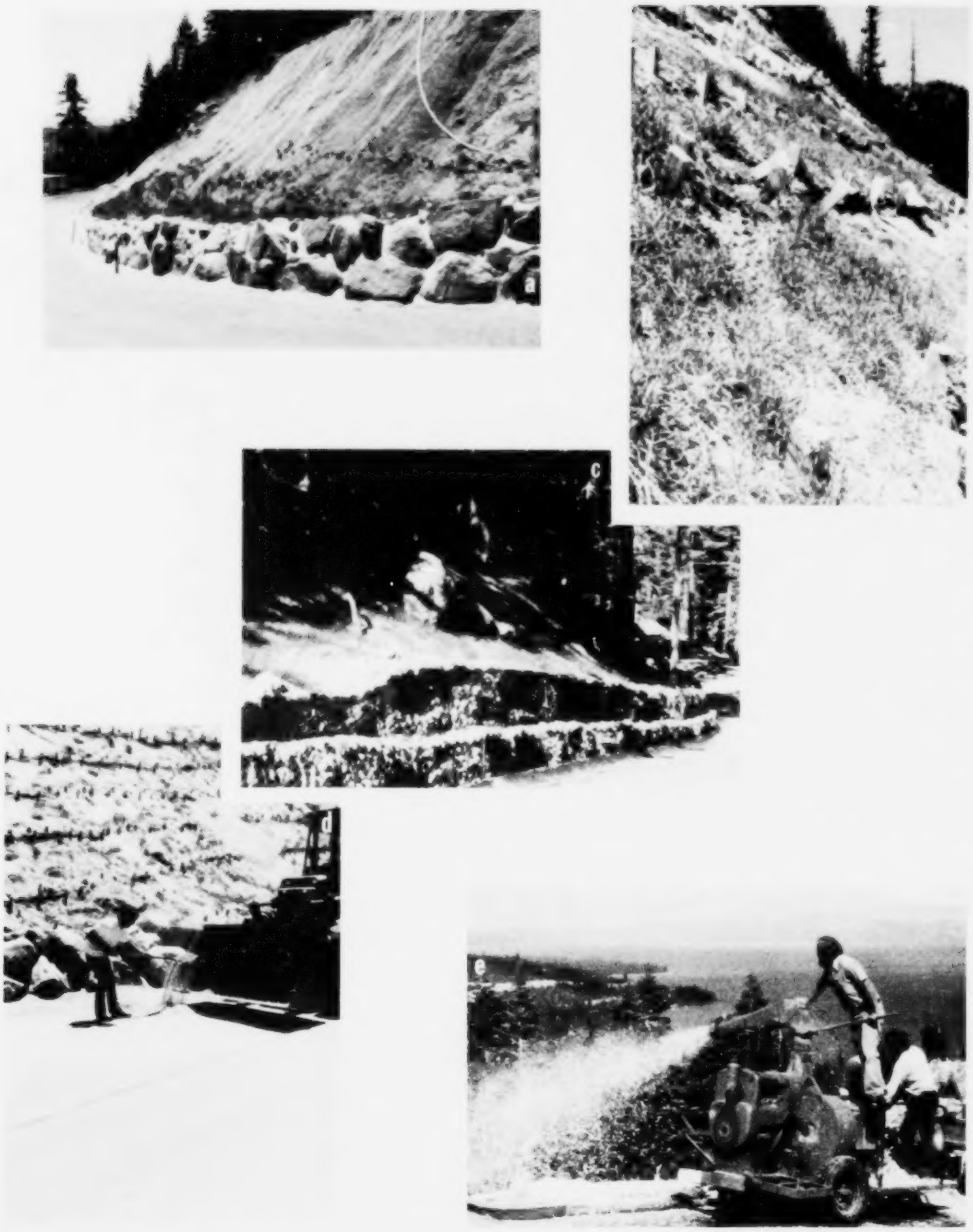


Figure 33. Erosion control measures at Site 2: (a) rock retaining wall, willow wattling and netting, (b) willow wattling, (c) gabion retaining wall and hydroseeded slope, (d) maintenance and cleanup operations near stabilized slope, and (e) application of straw mulch.

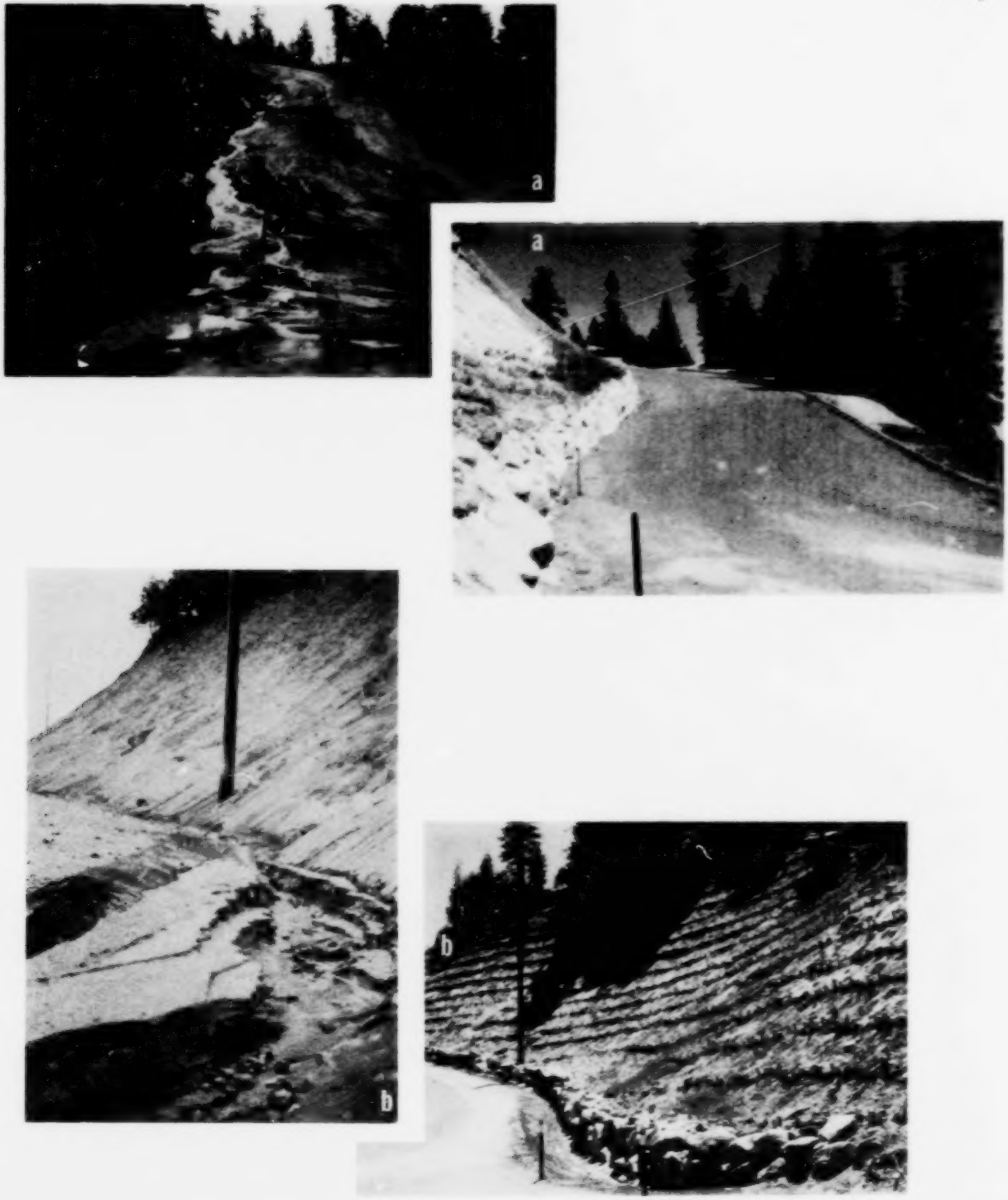


Figure 34. Conditions before and after implementation of erosion controls at Site 2: (a) and (b) heavily eroded, unstable road-cut slopes, and same slopes with erosion controls, including slope reshaping, retaining walls, willow wattling, hydroseeding, and netting.

## Table 53. ESTIMATED COST FOR PERMANENT SLOPE STABILIZATION METHODS [8, 10]

| Method | Units | Cost | | | Total cost |
|---|---|---|---|---|---|
| | | Materials | Labor | Equipment | |
| Rock retaining wall, 4 ft high | \$/lf | 5[b] | 12 | 10 | 27 |
| Redwood retaining wall, 3 ft high | \$/lf | 12 | 10 | 3 | 25 |
| Gabion retaining wall, 3 ft high | \$/lf | 10 | 8 | 3 | 21 |
| Slope bottom bench | \$/lf | none | 5 | 2 | 7 |
| Wattling | \$/lf | 0.3 | 1.9 | 0.1 | 2.3 |
| Slope steping | \$/acre | none | 320 | 250 | 570 |
| Slope serration | \$/acre | none | 300 | 120 | 420 |
| Overhang removal and scaling (manual) | $\$/yd^3$ | none | 39 | 2 | 41 |
| Overhang removal and scaling (backhoe) | $\$/yd^3$ | none | 5 | 2 | 7 |

a. ENR 3000.

b. Assumes use of native material.

## Table 54. ESTIMATED COST OF TEMPORARY SOIL STABILIZATION METHODS [8, 10]

| Method | Cost, \$/acre | | Total cost, \$/acre |
|---|---|---|---|
| | Materials | Labor and equipment | |
| Jute matting | 2,700 | 6,400 | 9,100 |
| Paper fabric | 2,900 | 6,600 | 9,500 |
| Plastic netting | 700 | 4,100 | 4,800 |
| Wood excelsior matting | 1,700 | 9,000 | 10,700 |
| Fiberglass roving | 1,900 | 2,100 | 4,000 |
| Hydromulching[b] | 500 | 700 | 1,200 |
| Chemicals and tackifiers | 400 | 200 | 600 |
| Wood chip application | 750 | 100 | 850 |
| Crushed gravel mulches | 620 | 180 | 800 |
| Straw mulch[c] | 150 | 530 | 680 |

a. ENR 3000.

b. 2.5 tons/acre.

c. 2.0 tons/acre.

### Table 55. SUMMARY OF UNIT COSTS OF RUNOFF MANAGEMENT SOURCE CONTROLS [8][a]

| Method | Unit costs, $/lf | | Total cost |
|---|---|---|---|
| | Materials | Labor and equipment | |
| Diversion dike | none | 4.24 | 4.24 |
| Runoff interception trench | none | 7.22 | 7.22 |
| Strawbale sediment barrier | 1.12 | 1.37 | 2.49 |
| Sandbag sediment barrier | 0.93 | 2.74 | 3.67 |
| Filter berm | 3.73 | 4.35 | 8.08 |
| Filter fence | 1.48 | 1.83 | 3.31 |
| Filter inlet | 0.62 | 1.68 | 2.30 |
| Siltation berm | 3.30 | 5.22 | 8.52 |

[a] ENR 3000.

### Table 56. ESTIMATED COST OF VARIOUS REVEGETATION METHODS [10][a]

| Method | Unit cost, $ | Cost, $/acre |
|---|---|---|
| Willow staking | 0.75/stake | 12,100 |
| Rooted shrub cuttings | 1.69/plant | 27,400 |
| Bare root seedlings | 0.78/plant | 13,100 |
| Seed with hydromulching | 0.19-0.28/$yd^2$ | 900-1,400 |
| Seed with tacked straw | 0.23/$yd^2$ | 1,100 |
| Seed with jute matting | 1.92/$yd^2$ | 9,300 |
| Seed with paper fabric | 2.00/$yd^2$ | 9,700 |
| Seed with excelsior | 2.25/$yd^2$ | 10,900 |
| Seed with straw and plastic net | 1.11/$yd^2$ | 5,400 |
| Seed with fiberglass roving | 0.83/$yd^2$ | 4,000 |

[a] ENR 3000.

## SOCIOECONOMIC IMPACTS

Land use planning and erosion control measures have improved the receiving waters in the Lake Tahoe area. Land use planning is considered essential in the initial stages of development, because of lower costs, greater aesthetics, and greater continuity of overall environmental protection. Erosion controls are also effective control alternatives for existing problems resulting from poorly planned or disturbed areas. Erosion controls are, however, more expensive to implement than equivalent results of land use planning. The cost of erosion controls used in developed areas is usually borne by local government or private landowners.

### Impacts on Private and Local Facilities

The costs to correct existing problems are greater than the costs associated with preplanned environmental controls during development. The costs to private landowners at Site 2 would be in excess of $2,500 per lot for the implemented erosion controls compared with approximately $360 per lot at Site 1, where extensive land use planning was used. The costs for the erosion controls represent a high financial burden for the individual landowner, and erosion controls are difficult to implement without public assistance.

Although the costs of environmental control in well planned developments are substantially lower to the individual lot owner, the developers are forced to make concessions in overall profitability by reducing the area of land available for development. This may increase the cost of individual lots, in addition to the costs for environmental planning and controls.

Because of the difficulty of enforcing regulations and environmental controls on private individuals in existing developments, local government agencies are often financially burdened by ill-fated developments, half-finished subdivisions, vacant lots, scarred land, and maintenance problems. The costs to the county for eroded sediment cleanup and maintenance within Site 2 have been estimated at $14,300 per year (ENR 3000) [10]. As a result of the erosion controls, massive cleanup efforts and costs to the county following rainstorms and snowmelt have been substantially reduced. The total cost of the erosion control work could be amortized (at 8%) over a 12.5 year period at the annual budget spent for cleanups [10].

Land values and demographics are affected by overall land use planning. Land use intensity controls, such as providing for open space, resulting in increased attractiveness to build (aesthetics), plus a reduction of available land to build on, can increase land values and the rate of development.

Long-term socioeconomic impacts can include a net population decrease due to rezoning and management of high erosion hazard lands and stream environment zones. Projections of ultimate potential population reductions in the Lake Tahoe Basin have been made: (1) a reduction of about 2,400 persons through rezoning, and (2) a maximum reduction potential of 35,000 persons through management of stream environment zones [2].

### Public Acceptance

A survey of 26,000 property owners in the Lake Tahoe Basin identified water pollution, scenic destruction, and too much commercialism as priority problems [2]. One of the main objections was high density developments. Environmental planning through land use control addresses most of these concerns.

Several beneficial uses of the waters in the Lake Tahoe Basin that can be adversely affected by poor land use planning and erosion sediment loadings have been identified and include [10]:

- Domestic water supply
- Agricultural water supply
- Water contact recreation
- Nonwater contact recreation
- Fresh water habitats
- Fish spawning

These considerations, together with the aesthetic improvement to scarred and disturbed areas through erosion control and land use planning, may be taken as positive public acceptance.

## Aesthetics

Aesthetics is probably the single most important factor for development and tourism in the Lake Tahoe Basin. It has contributed to both the economic growth of the area and its environmental problems. Land use planning provides for open space and controls development. Erosion controls can improve scarred and disturbed areas by grooming and revegetation of the land. Mechanical stabilization methods, such as retaining walls, may be considered aesthetic improvements in themselves. Erosion controls can also prevent sediment loads from entering streams, allowing the stream to clean and scour itself.

However, aesthetics is one of the main reasons for the problems developing in the first place. The uncontrolled high density subdivision on the steep terrain at Site 2 was developed to take advantage of the spectacular view afforded by the location, and was probably an influencing factor in the marketability of the property.

# SECTION 7

## MANAGEMENT OF A NATURAL DRAINAGE SYSTEM
## THE WOODLANDS, TEXAS

The planners of a new community, The Woodlands, in southeastern Texas, attempted to minimize the water resources problems traditionally caused by urbanization. The governing principle of this development's planning was preserving both the natural drainage system and the predevelopment surface water-groundwater balance. In addition to avoiding many of the problems of urban runoff, the planners also hoped to decrease the site development costs and create a unique new community with a natural forest setting that would attract home buyers.

Urbanization of an undeveloped site changes both the quantity and quality of runoff. The primary factors increasing the quantity of runoff are: (1) the replacement of porous soils by impervious pavement and building sites, and (2) a decrease in the infiltration of ponded water caused by both grading and the replacement of natural drainage systems by storm sewers or lined ditches. The change in runoff quality is due to the introduction of pollutants related to an urban setting. Construction activity will increase suspended solids loadings; intensive landscaping will increase nutrient, pesticide, and herbicide concentrations; litter will increase the quantities of floating solids; and various nonpoint sources will increase the concentrations of trace metals, bacteria, and oil and grease.

At The Woodlands an effort was made to maintain runoff quantities at a predevelopment level by planning to avoid site conditions that increase runoff. It was hoped that the techniques used to prevent increased quantity would also preserve the quality of runoff at pollutant concentrations close to that of predevelopment conditions. The runoff quality had to be acceptable for recreational lakes, irrigation, and groundwater recharge.

Studies were conducted by a team from Rice University to determine how effectively The Woodlands' development preserved the water resources of the site and to determine if changes in runoff water quality would make the surface water unacceptable for recreational or aesthetic uses [1].

### PROJECT DESCRIPTION

The Woodlands project began in 1971 with $50 million in loans guaranteed by the U.S. Department of Housing and Urban Development. The project was one of 13 new communities nationwide begun under a 1970 federal program to encourage the planning of self-contained, economically balanced urban centers that would demonstrate modern planning concepts. The natural drainage concept was a part of The Woodlands planning process from the beginning and extensive surveys of soils, drainage, vegetation, and water table levels were made to provide background data for the drainage planning.

## Site Development

The Woodlands is a 7,300 ha (18,000 acre) site approximately 40 km (25 miles) north of Houston, Texas. The predevelopment vegetation at the site consisted of a pine and oak forest with generally medium-to-heavy understory vegetation. The site is flat with slopes commonly from 0 to 3% and predevelopment stream bed slopes averaging 0.2%.

### Soils--

The soil types, mapped before the initial development of The Woodlands, are fine sands and fine sandy loams underlain by an impermeable clay zone at a depth ranging from 0 to 2 m (0 to 80 in.) below the ground surface. The clay zone supports a perched water table at 0.4 to 2 m (15 to 80 in.). The soils were categorized by the depth to the impermeable layer and thus the ability to store storm runoff in the upper soils.

The shallow depths of permeable soil, the high water table, and the flat terrain of the site indicate that casual water ponding is common during rainy periods.

### Predevelopment Drainage--

The major portion of the site drains into either Panther Branch or its main tributary, Bear Branch. Panther Branch and the remainder of the site are tributary to Spring Creek and ultimately Lake Houston, which is a surface water supply for the Houston metropolitan area. The three streams are shown in Figure 35 as they relate to The Woodlands. Panther and Bear branches are meandering streams with well-defined, low flow channels and very wide, flat flood plains. The low flow channel varies between 1.5 and 6 m (5 and 20 ft) wide and has several pools. The channel increases from approximately 1 m (3 ft) at the headwaters to 3 m (10 ft) at the junction with Spring Creek.

The average yearly rainfall for the north Houston area is approximately 115 cm ( 46 in.) from a combination of intense summer thunderstorms and more prolonged winter rains. It is estimated that 10 to 15% of the rainfall becomes runoff in undeveloped portions of The Woodlands; the remainder will pond, then evaporate or infiltrate [1].

## Problem Assessment

The challenge at The Woodlands was to develop a self-contained, new community while maintaining the preconstruction quality of the land, water, and forest. Obviously, trees would have to be cut for structures and roadways, but there would not be a general clearing of tracts or lots. The drainage system should blend in with the natural setting and yet serve the developing area.

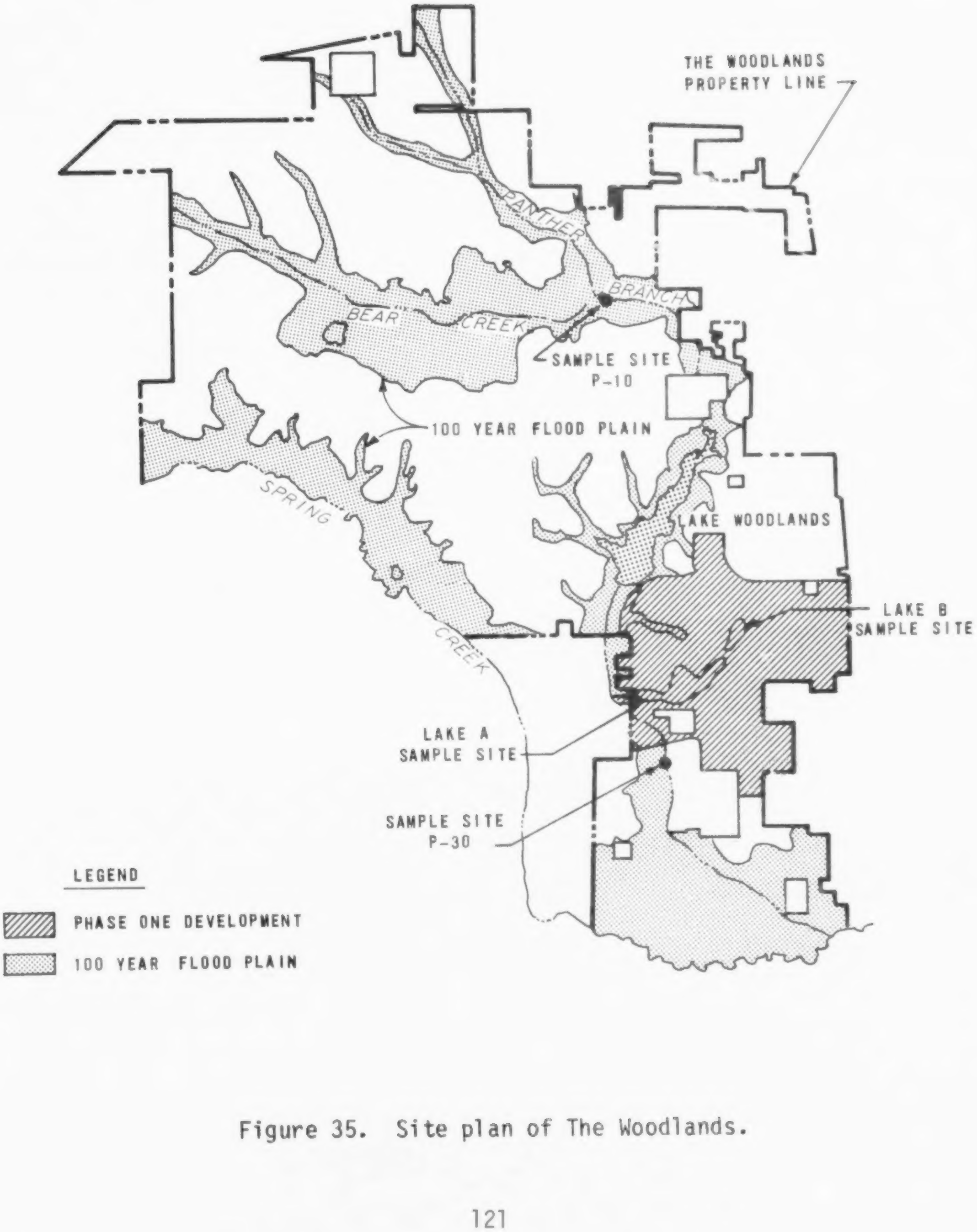


Figure 35. Site plan of The Woodlands.

Two major problems were faced in designing the drainage system for The Woodlands. First, the groundwater at the site had to be protected and preserved and second, the surface water had to be suitable for onsite use and maintained both to preserve the quality of Lake Houston and not cause downstream flooding.

### Groundwater--

There are two groundwater reservoirs that are important to the development of The Woodlands. The deeper reservoir provides the drinking water for the new community; the need for preserving its quality and quantity is obvious. The shallow reservoir is the perched water table found within 3 m (80 in.) of the surface. The existing vegetation of The Woodlands has developed an ecological balance based on this high water table.

Lowering the level of the water table would damage or at least change the balance and create a new vegetation ecosystem. Conventional development calls for the interception of surface runoff and its removal from the vicinity of housing areas. Although central ponds or recharge wells could preserve the deeper groundwater reservoir, local drainage would mean the loss of perched water in any area drained by storm sewers. The problem at the Woodlands was to preserve both levels of groundwater throughout the site.

### Surface Water--

Preservation of the surface water quality was required both for onsite recreation and aesthetics and for downstream uses. Development plans call for small recreational lakes in the neighborhoods and a large impoundment on Panther Branch to be called Lake Woodlands. The primary purpose of these ponds and lakes is to increase the recreational appeal of the new community and enhance the aesthetic appeal of the housing sites near the ponds and lakes. However, it would be desirable for the lakes to serve as part of the water management system by storing stormwater peaks and providing a source of nonpotable water for irrigation and recharge. The quality has to be preserved to serve all of these goals.

Urban runoff is normally contaminated by significant amounts of pollutants, including solids, oxygen-demanding substances, nutrients, pesticides, herbicides, trace metals, and floating litter or grease. A major goal of The Woodlands was to minimize the presence of these pollutants, then study the runoff to understand the impact of the remaining pollutants on the water resources system. Potential problems include the following:

- The growth of objectional lake algae or water plants fertilized by nutrients in runoff from golf courses and other landscaped areas.
- A buildup of toxic pesticides, herbicides, or metals in the lakes that could kill fish and pollute groundwater resources.
- An increase of the bacteriological indicator organisms in the lakes to levels that would require prohibition of recreational activities.

- A large increase in the quantity or a decrease in the quality of water in Spring Creek due either to storm runoff or sewage treatment effluent.

It was expected that any design would cause some decrease in surface water quality--the problem was to develop and modify a system that would keep the impact of the pollutants to a level acceptable for the planned uses of the surface water.

## Countermeasure Philosophy

The water management planners at The Woodlands assumed that the predevelopment stream system provided good quantity and quality control and that a channelized urban system would not serve all the planning goals. The resultant plan was to preserve the natural drainage system as much as possible. The development was to be managed so that sites chosen for buildings, roads, and other construction would have a minimum impact on the natural drainage and that unavoidable impacts would be offset by improvements to neutralize these impacts. The improvements would not be the common storm sewer system designed to remove water but rather ponding systems designed to infiltrate runoff.

An interrelated planning goal was the desire to achieve a hydrologic balance at the site by recharging the deep groundwater reservoir at the same rate as water is withdrawn for the potable water system. The recharge would result from infiltration of water through the lake systems. The lakes, in turn, would be fed by storm runoff and sewage treatment plant effluent. A key point to achieving this goal is an acceptable quality of sewage effluent so that the lakes can be maintained during low rainfall periods.

## Implemented Countermeasures

### Planning--

The first step in preserving the natural drainage system was an extensive survey of vegetation, soils, and slopes at the site. The purpose was to indicate portions of the site suitable for various housing densities, shopping centers, community centers, and roads. Each hydrologic subbasin was to retain enough pervious soil to infiltrate the runoff from a 2.54 cm (1 in.) storm. The planned dwelling unit densities of from 2.5 to 37 per ha (1 to 15 per acre) required site imperviousness ranging from 24 to 56% and site clearing from 37 to 93%. The high density developments were limited to locations where the soil was already naturally impervious and the vegetation considered less valuable.

### Existing Drainage--

The existing drainage system was protected by prohibiting development within the 25 year flood plain of tributary streams and controlling development within the 100 year flood plain of the major streams. This was a major commitment of land since approximately one-third of the site lies within the protected flood plains of Bear Branch, Panther Branch, and Spring Creek. An important part of the ability to maintain flood plains, open space, and drainage channels at The Woodlands was the location of a 36-hole golf course. The course winds in and out among the neighborhoods using land unavailable for housing because of planning restrictions.

Every effort was to be made to preserve the tributary swales in their natural, forested condition. The planners considered that excessive use of even broad, shallow (10:1), grassy, man-made swales would endanger the perched water table and consequently the site's ecological balance. The forested swales were considered ideal for percolation and the prevention of soil erosion.

### Natural Drainage--

In areas where the existing drainage had to be improved because of development, natural systems were constructed. Natural systems, as defined at The Woodlands, are broad, grassy swales, instead of lined ditches or drain pipes, and check dams with temporary ponding rather than permanently retained water. The check dams were expected to maintain local perched groundwater and yet dry out often enough to prevent mosquito breeding.

Infiltration into the deep groundwater aquifer was to be maintained by large multipurpose lakes servings as recreational facilities, stormwater retention basins, and recharge facilities. These lakes would be expected to offset the quantity of water withdrawn from deep aquifers that supply the potable water requirements of the residents. The lakes would also serve the drainage system as peak flow retention basins to prevent storm flow surges in to Spring Creek. It is anticipated that in spite of the best efforts to maintain infiltration and percolation at the site, there would be increased runoff during heavy storms. The lake systems would be able to capture the surge, alleviating downstream flooding and allowing some pollutants to settle out.

### Other Countermeasures--

Two concepts expected to become part of the future planning at The Woodlands are the reuse of sewage effluent and the construction of porous pavement to preserve infiltration in locations where large parking lots are required. Both concepts were to be investigated. An attempt was to be made to determine the quality of treated effluent suitable for recycling to surface waters by investigating disinfection, algae blooms, and the relative quality of lakes, runoff, and reclaimed water. Porous pavement was to be tested for maintenance of porosity and quality of the water draining through the pavement.

The final countermeasure to be used at the development was to control the use of fertilizers, herbicides, and pesticides by making them unnecessary. The planners hoped that by leaving the natural plant and animal life in the predevelopment ecological balance, chemical substances would not be needed to promote or control plant growth. Limits were placed on the amount of clearing by developers and homeowners are encouraged to leave existing brush and trees instead of planting lawns.

## PERFORMANCE

An evaluation of the performance of countermeasures at The Woodlands is based on data collected by investigators from Rice University during the period January 1974 to April 1976 [1]. Data were collected both at The Woodlands and at two other Houston watersheds that serve as examples of fully developed urban neighorhoods. The first watershed, Hunting Bayou, is a 800 hectare (2,000 acre) area in northeast Houston. It is 48% residential and 46% commercial-industrial; 28% of the area has storm sewers. Westbury, the second control watershed, is an 80 hectare (200 acre) area in southwest Houston that is 100% residential and 100% storm sewered.

During the studies, the population of The Woodlands was approximately 2,000 as compared with the originally predicted population of 150,000 by 1992. Extensive construction was taking place at the site including excavation of a borrow pit that will become a major lake. Therefore, the results reflect only a small portion of the final development and are biased by construction activity.

Samples were collected from four principal locations at The Woodlands, which are shown in Figure 35 and described as follows:

- Station P-10 is near the confluence of the Bear and Panther Branches. The tributary area is 6,500 hectares (16,000 acres) of natural forest.
- Station P-30 is on the Panther Branch downstream of the Phase One development. The area includes 8,700 hectares (21,500 acres) that is 90% forest and 10% developed or developing.
- Lake B is located at the inlet to Harrison Lake. The 135 hectare (355 acre) tributary area was undergoing development.
- Lake A is located at the outlet to Harrison Lake. The 195 hectare (485 acre) area includes the Lake B tributary area and some areas directly tributary to the lake.

### Runoff Quality

The runoff quality data from January 1974 through April 1976 for dry-weather samples and samples taken during 17 storm events are presented in Table 57. Regression analysis of the runoff at four sites was used to find mean loadings in kilograms per hectare (pounds per acre) of pollutants for 2.54 cm (1.0 in.) of runoff. The values, 95% confidence limits, and ranking of sites are shown in Table 58. Several observations can relate the sampling data to land use.

- Suspended Solids - The most significant solids loading appears to come from the construction activity upstream from Station P-30. The Hunting Bayou area shows solids pollution from commercial-industrial areas and barren urban land. The fully developed residential Westbury has approximately the same solids concentration as the forested Station P-10. The conclusion is that suspended solids pollution from residential areas will be a problem only during construction.

## Table 57. RUNOFF QUALITY [1]
mg/L

| | No. of storms | Suspended solids | | Total COD | | Total phosphorus | | Kjeldahl nitrogen | |
|---|---|---|---|---|---|---|---|---|---|
| | | Range | Average | Range | Average | Range | Average | Range | Average |
| Dry-weather flow | | | | | | | | | |
| The Woodlands - P-10 | -- | -- | 23 | -- | 50 | -- | 0.06 | -- | 0.93 |
| The Woodlands - P-30 | -- | -- | 81 | -- | 51 | -- | 0.14 | -- | 1.67 |
| Storm flows | | | | | | | | | |
| The Woodlands - P-10 | 8 | 7-67 | -- | 43-63 | -- | 0.03-0.09 | -- | 0.10-1.61 | -- |
| The Woodlands - P-30 | 12 | 109-321 | -- | 40-51 | -- | 0.13-0.30 | -- | 0.06-1.41 | -- |
| The Woodlands - Lake B | 8 | 283-2,880 | -- | 49-123 | -- | 0.11-0.53 | -- | 1.79-4.14 | |
| The Woodlands - Lake A | 8 | 24-245 | -- | 39-54 | -- | 0.73-1.14 | -- | 1.48-2.19 | -- |
| Westbury | 2 | 24-70 | -- | 39-54 | -- | 0.73-1.14 | -- | 1.48-2.19 | -- |
| Hunting Bayou | 5 | 71-207 | -- | 77-179 | -- | 0.41-1.28 | -- | 1.56-3.94 | -- |

## Table 58. POLLUTANT LOADINGS FROM RUNOFF [1]

| | | Rank (Decreasing pollution ⟶) | | | |
|---|---|---|---|---|---|
| Pollutant | | 1 | 2 | 3 | 4 |
| Suspended solids | Area | P-30 | HB | WB | P-10 |
| | Mean lb/acre-in. | 43 | 38 | 14 | 8.2 |
| | Confidence limits | ±18 | ±5 | ±74 | ±2 |
| COD | Area | HB | P-10 | P-30 | WB |
| | Mean lb/acre-in. | 19 | 14 | 13 | 9.5 |
| | Confidence limits | ±7 | ±1 | ±1 | ±24 |
| Soluble COD | Area | P-10 | P-30 | HB | WB |
| | Mean lb/acre-in. | 10 | 9 | 4.4 | 4.1 |
| | Confidence limits | ±1 | ±1 | ±2 | ±28 |
| Total phosphorus | Area | HB | WB | P-30 | P-10 |
| | Mean lb/acre-in. | 0.28 | 0.24 | 0.021 | 0.014 |
| | Confidence limits | ±0.12 | ±0.66 | ±0.007 | ±0.003 |
| Kjeldahl nitrogen | Area | HB | WB | P-30 | P-10 |
| | Mean lb/acre-in. | 0.95 | 0.40 | 0.30 | 0.28 |
| | Confidence limits | ±0.10 | ±1.10 | ±0.06 | ±0.10 |
| $NO_3$ | Area | WB | HB | P-30 | P-10 |
| | Mean lb/acre-in. | 0.088 | 0.037 | 0.020 | 0.012 |
| | Confidence limits | ±0.032 | ±0.013 | ±0.018 | ±0.008 |

Note: P-10, P-30 - The Woodlands
HB - Hunting Bayou (Houston)
WB - Westbury (Houston)

$$\frac{\text{lb}}{\text{acre-in.}} \times 2.27 = \frac{\text{kg}}{\text{ha-cm}}$$

- Organics (COD, soluble COD) - The data indicate that organic pollutants should decrease as the forested area is developed. The exception is the high particulate COD at Hunting Bayou when compared with all other stations. This pollution is probably due to oils, grease, and litter dumped in the open channels and will probably not be as apparent in the more residential woodlands.

- Nutrients - The nutrient samples indicate where the most significant problems will develop. Both developed areas, Hunting Bayou and Westbury, show nutrient levels that are much higher than the stations at The Woodlands. Levels increase as land use evolves from forested to developing residential, to residential, to mixed urban.

The runoff sampling data show that nutrients are a problem and if the attempt to decrease the use of fertilizers at The Woodlands is successful over the long term, a major problem with urbanization will be mitigated. However, considering the long-term construction period at the site, it appears that planning should also consider interim controls of suspended solids.

### Effect of Lake Impoundment

One of the drainage planning concepts was to use the major lakes at the development to absorb stormwater surges and equalize the concentrations of pollutants in runoff. An analysis of flow and pollutants into and out of the lake system during a 1975 storm is shown in Table 59 and Figure 36. The results indicate that the lakes are an effective sediment trap, reducing the suspended solids loading by 80%. However, the results show some enrichment in the lakes for certain forms of nitrogen and phosphorus. The reason for the nutrient enrichment was not determined, but is probably due to a combination of factors including: (1) direct runoff from areas adjacent to the lake, (2) direct nutrient-enriched rainfall, and (3) high concentrations of these nutrient forms in the lake before runoff began.

Table 59. WATER QUALITY ANALYSIS
OF THE WOODLANDS LAKE SYSTEM DURING A 1975 STORM [1]
mg/L

| | Influent | | Effluent | |
|---|---|---|---|---|
| Parameter | Average | Maximum | Average | Maximum |
| Orthophosphate | 0.005 | 0.013 | 0.015 | 0.048 |
| Total phosphate | 0.11 | 0.36 | 0.10 | 0.19 |
| Ammonia | 0.11 | 0.15 | 0.16 | 0.26 |
| Nitrate | 0.15 | 2.1 | 0.28 | 0.32 |
| Kjeldahl nitrogen | 1.86 | 3.1 | 1.3 | 2. |
| Suspended solids | 1273 | 2660 | 245 | 356 |
| Total COD | 63.7 | 87.0 | 41.8 | 45.0 |
| Soluble COD | 32.0 | 45.0 | 26.4 | 31.0 |

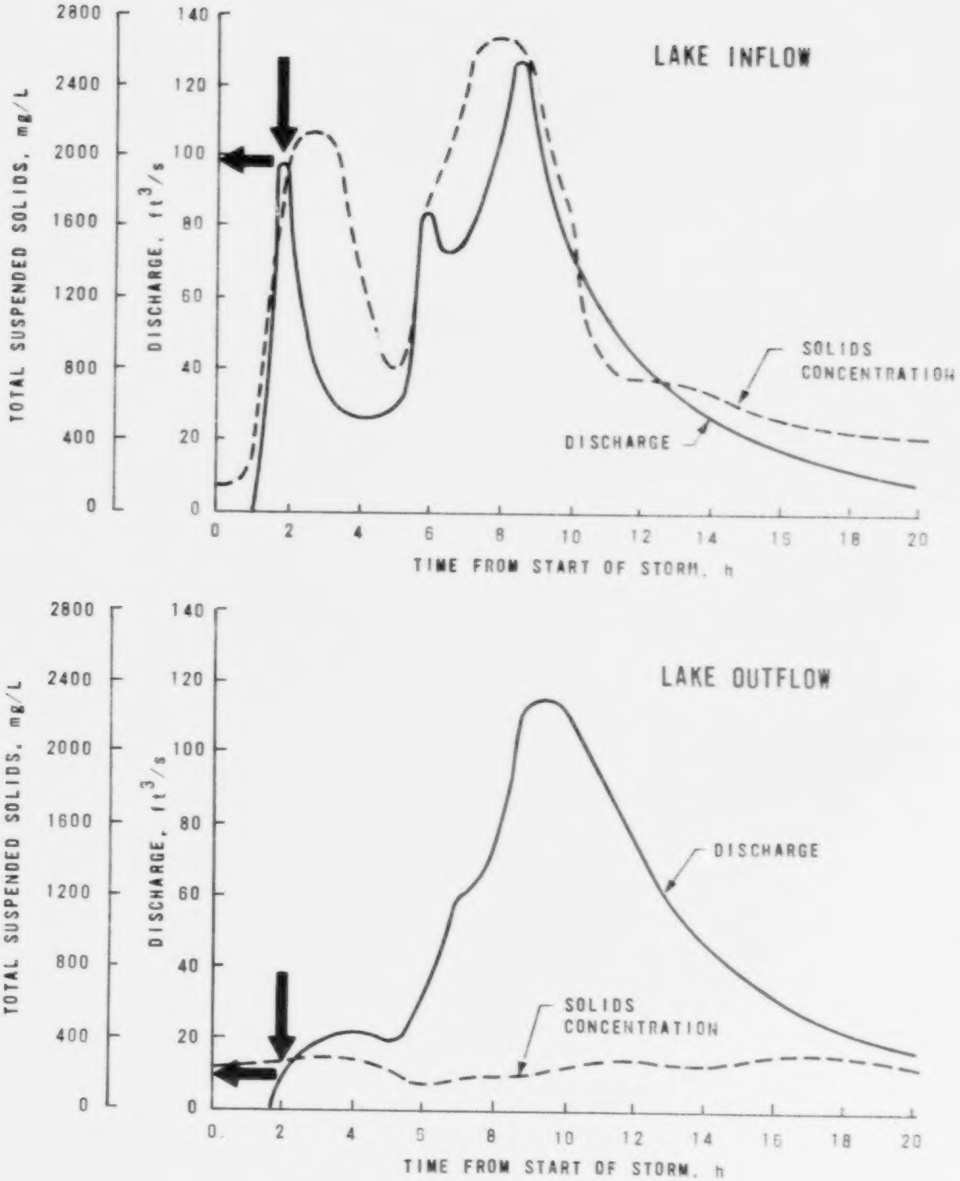


Figure 36. Effect of lake impoundment on storm flowrates and suspended solids concentration [1].

The peak flow out of the lakes was only slightly lower than peak inflow; however, the storm was quite large, 10 cm (4 in.) of rain in 10 hours. The first peak of the storm, in which 5 cm (2 in.) fell in 3 hours, was effectively controlled as the influent peak of 2,800 L/s (100 $ft^3/s$), at about 2 hours, was reduced to 550 L/s (20 $ft^3/s$) at discharge.

Using lakes for dampening hydraulic and pollutant peaks appears to be effective, particularly for suspended solids.

### Porous Pavement

A parking lot was built at The Woodlands conference center to test porous pavement. The lot had two sections, one-half conventional pavement and one-half porous pavement with a sand and gravel underdrain.

Tests indicated that runoff would penetrate the porous pavement and build up in the underdrain creating a slowly draining reservoir. The result was similar to a detention pond: runoff was delayed and peaks dampened [2].

A comparison of runoff quality between two sections of the lot showed that organic and nitrate levels were lower in the runoff from porous pavement, while ammonia levels were higher. The investigators suggest that anaerobic decay may be taking place in the underdrain. Lead and zinc concentrations were much lower in the runoff from porous pavement. Lead averaged 0.05 mg/L, while zinc was 0.18 mg/L [2].

The safety and driveability of the porous pavement was satisfactory; however, it was susceptible to clogging and will require periodic vacuum sweeping and high pressure washing to remain permeable. Maintenance can probably be decreased by better control of dirt carried onto the lot by construction vehicles.

## IMPACTS

The water resources plan for The Woodlands will succeed only if the effects of the natural drainage system are acceptable to the residents of the community. Investigation of the environmental impacts centers on the multipurpose lakes and whether they can maintain an acceptable water quality while receiving storm runoff and sewage treatment effluent. The socioeconomic impact concerns the willingness of the residents to accept the community as planned with both restricted forest clearing and several high density townhouse developments needed to provide an overall community density.

### Environmental Impacts

Several studies have been conducted to examine potential problems in the lake system [3-6]. The investigations included: (1) the eutrophication potential in the lakes, (2) chlorine and ozone toxicity as applied to the disinfection of wastewater recycled to the lakes, (3) bacterial characteristics of the runoff, and (4) organochlorine compounds in the runoff and lakes.

### Eutrophication Potential--

Nutrient enrichment in a lake system can speed the natural eutrophication process and cause unwanted algal blooms. A large algae population is aesthetically unpleasing, interferes with recreation, and may cause fish kills. In view of the expected increase of nutrients in the runoff as The Woodlands develops, a study was made of the effects of nutrient addition on algae growth at the site.

Samples taken from Panther Branch, the lake system, Hunting Bayou, and Westbury were spiked with nitrogen or phosphorus or both and the algae growth was observed. In most cases, the low flow conditions were phosphorus limited and storm flow conditions were nitrogen limited.

Recommendations from this study include: (1) sewage treatment effluent should have phosphorus removed before it is recycled to the lakes, (2) there is no reason to remove phosphorus during storms, and (3) the "first flush" of a storm should be diverted or treated to prevent nutrients from entering the major lake systems [3].

### Disinfectant Toxicity--

Recyling of wastewater effluent to a recreational lake will require adequate disinfection. The residual levels of disinfectant in the lake system may be a problem to the fish population. A study was conducted to examine chlorine and ozone residuals in connection with sewage treatment at The Woodlands.

The results of the study with channel catfish showed that in a 96-hour bioassay the $LC_{50}$ for chlorine is 0.07 mg/L and for ozone is 0.03 mg/L. Therefore, based on the "Aquatic Life Water Quality Criteria" of 1/10 value, the acceptable level for chlorine is 0.007 mg/L and ozone should be below detection levels [4].

### Bacterial Characteristics--

An important part of the multiuse concept for The Woodlands' lakes is their recreational value. A test of the suitability of the lakes for both stormwater control and recreation is the ability to meet current Texas standards for acceptable bacteria levels. The standard is a 30-day mean fecal coliform level of less than 200 organisms/100 mL for water contact recreation and 2,000/100 mL for noncontact recreation. Mean values for a number of samples taken at different sites and under different conditions are shown in Table 60.

The data from P-10, P-30, and Westbury show a trend of deterioration with urbanization and high levels of bacteria even in the rural stream represented by Station P-10. It appears doubtful that the lakes would be acceptable for contact recreation. Since the standard is based on a mean of five samples in 30 days, it would include both storm and low flow periods, and the lakes would probably be acceptable for noncontact recreation. An interesting result is the apparent settling and die-off of coliform in the existing lake system. During storm periods, coliform levels are an order of magnitude lower in the lake effluent than in the influent.

Table 60. MEAN VALUES OF FECAL COLIFORMS [5]
Organisms/100 mL

| Site | Condition | Fecal coliform/100 mL |
|---|---|---|
| P-10 | Low flow | 135 |
| P-30 | Low flow | 240 |
| Lake B | Low flow | 95 |
| Lake A | Low flow | 35 |
| P-10 | Storm | 1,000 |
| P-30 | Storm | 2,950 |
| Westbury | Storm | 24,500 |
| Lake B | Storm | 2,040 |
| Lake A | Storm | 220 |
| Chlorinated sewage | -- | 18-42 |

A concurrent study of soil and sediment found the following levels of coliforms:

| | Fecal coliforms/gram |
|---|---|
| Stream sediment | 20-20 |
| Soils (sampling stations) | 20-40 |
| Lake sediment | 20-60 |
| Soils (golf courses and swales) | 20-280 |

Leaching and scouring of the soil could have contributed to the levels of coliform in the runoff.

Finally, disinfection studies were performed to determine concentrations required to lower fecal coliform levels. Chlorine concentrations of from 8 to 16 mg/L were required to satisfy disinfectant demand in samples from Station P-30 and 10 mg/L in samples from Lake A. An ozone dose of 32 mg/L was required to disinfect a Lake A sample [5].

### Organochlorine Compounds--

Another project was undertaken to make a preliminary study of the organochlorine compounds in the water system of the developing community. Emphasis was placed on the examination of soil, water and fish for concentrations of polychlorinated biphenyls (PCBs) and chlorinated hydrocarbon pesticides.

An unexplained rise in PCB concentrations was observed in the first half of 1974 at most of the sample sites. Peak concentrations reached 341 parts/billion (ppb) in the soil and 8.2 ppb in the water. In August and for the remainder of the study, the soil samples returned to a base level of from 1 to 2 ppb and the water less than 0.5 ppb. The 6 months of high values were not repeated the following year. The investigators speculated that since the high levels of PCBs coincided with a period of cut and earth moving, an abandoned landfill may have been disturbed [6].

Chlorinated hydrocarbon pesticides found in Panther Branch fish included trace amounts of DDE both upstream and downstream and trace quantities of dieldrin upstream, while 2.1 ppb of dieldrin were found in fish at Station P-30. The Woodlands golf course was also investigated for pesticide residue; Mirex, a pesticide used for fire ant control, was discovered. The concentration ranged from 5 to 15 ppb in water, 15 to 30 ppb in soil, and 30 to 55 ppb in mosquito fish. Chloradane was also found in golf course runoff and again showed amplification in the fish, with 10 to 40 ppb in crayfish, while only 5 to 20 ppb were found in soil and water.

#### Environmental Summary--

The studies of the environmental impacts caused by the water management system bring out several important points for management of the lakes:

1. Preservation of aesthetic values by control of eutrophication can be most easily accomplished by flushing and dilution with low phosphorus water during low flow periods. Treated wastewater or well water will be acceptable.

2. Maintenance of fish in the lakes will require that recycled wastewater or disinfected stormwater have very low residual chlorine or ozone concentrations.

3. Swimming and other water contact recreation would require disinfection of runoff since even upstream water quality is of only marginal bacterial quality. Boating and noncontact recreation can be maintained especially in the downstream unit of double lake systems, such as Harrison Lake (Lakes A and B).

4. Pesticides used at the site will contaminate fish and, if recreational fishing is allowed, pesticides will have to be controlled and fish sampled periodically. Lakes constructed downstream of areas using pesticides (golf courses) may have restricted fishing.

### Socioeconomic Impacts

The socioeconomic questions at The Woodlands are: (1) will homebuyers accept a natural drainage system with swales and requirements for leaving part of their lots in a natural condition, and (2) can the quantity of open space, flood plain reserves, and lakes envisioned be maintained and still allow the developers to profit.

### Acceptance of Natural Drainage and the Forest Setting--

A series of photographs of The Woodlands are shown in Figures 37 and 38. The drainage system is shown in Figure 37 and various examples of the forest setting at The Woodlands are shown in Figure 38. In almost all cases, the individual homesites are being maintained with more natural vegetation than similar developments in north Houston. Forested landscaping is encouraged by a monthly award for best natural landscaping and by sales brochures emphasizing the forest setting and, thus, attracting homebuyers interested in the concept. The natural setting has become an identifying point for the community and both homeowners and developers are interested in maintaining the idea. The only legal control over clearing at the site is a deed restriction requiring a permit to cut trees that are in excess of 15 cm (6 in.) in diameter.

The natural drainage system itself does not receive the same public attention given to natural landscaping. Homeowners complain only if their property floods, which has not been a significant problem. The developers believe that the use of a natural system has saved a great deal of initial capital cost and indicate that maintenance has not been a problem [7]. The best indication of the success of the plan is that the developers intend to follow the same concepts in the future development at The Woodlands.

### Open Space--

The development has maintained a significant amount of open space around the drainage system and the golf course. Present plans are to continue the existing drainage concepts and offset the cost of land committed to flood plains by savings in the initial cost of natural drainage. The next area of development contains a large recreational lake that will be the focal point of the area. The original plans for development had included high density townhouse and apartment areas to offset the open space and provide an overall density that would be profitable to the developers. In the first years of the community, the townhouses did not sell well because of a homeowner preference for single-family housing. However, there has been a recent upsurge in the demand for townhouses, and another section of high density development will be built.

### Socioeconomic Summary--

In any housing development, the values and opinions of the homebuyer will eventually prevail. An appealing environment will raise home values, but the buyer decides what characteristics are appealing. The natural landscaping at The Woodlands is apparently successful and will be carried into subsequent development phases. Economics now favor natural drainage, lake systems, and open space. However, if economics should reverse and favor a denser, more conventionally sewered site, the environmental issues would probably not outweigh the economics.

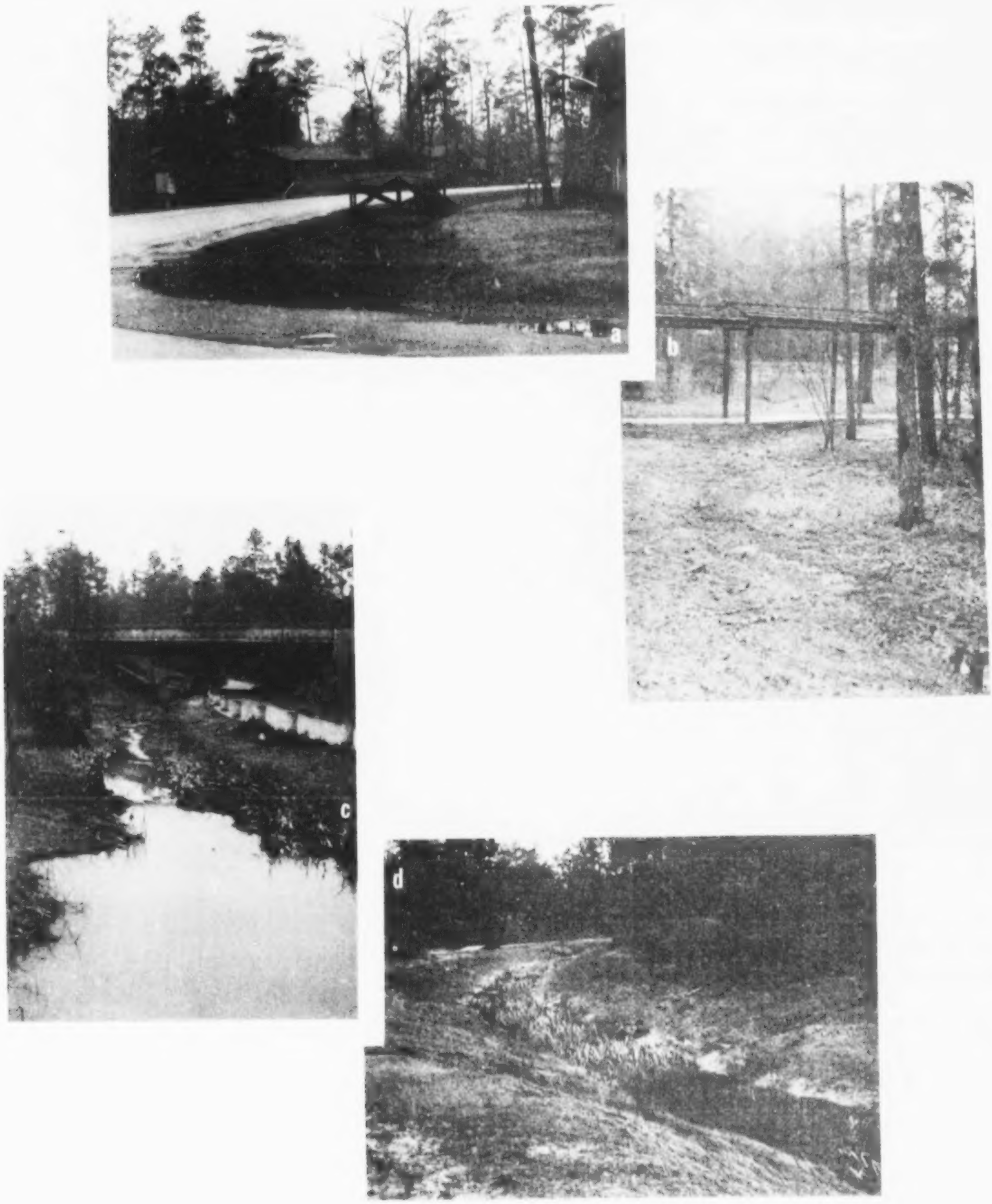


Figure 37. Natural drainage system at The Woodlands: (a) roadside swale in a residential neighborhood, (b) area drainage at the conference center, (c) influent to Lake Harrison, and (d) major drainage stream.

Figure 38. Landscaping with natural vegetation at the Woodlands: (a) office building and (b) and (c) the front yards of award winning homes.

# SECTION 8

# BEST MANAGEMENT PRACTICES
# ORANGE COUNTY, FLORIDA

The primary water quality problem in Orange County is the degradation of the many lakes and waterways; approximately one-tenth of Orange County's 2,590 $km^2$ (1,000 $mi^2$) is surface water. Stormwater runoff is depositing nutrients and sediments in the nearly 1,100 waterways and lakes, causing an increased rate of eutrophication. An areawide strategy has been developed to reduce the stormwater runoff loadings by implementing BMPs.

The BMPs used in Orange County include nonstructural and low structural facilities to control and/or treat stormwater runoff near its source. The BMPs in Orange County have demonstrated the effectiveness of individual unit controls to reduce the volume and pollutant loadings of stormwater runoff to the receiving waters.

The long-term environmental and socioeconomic impacts of the BMPs have not been fully identified because they have only been implemented recently, but the proposed program studies have projected the impacts of full program implementation. The BMPs have effectively reduced flooding hazards and have provided multiuse facilities for recreation and/or aesthetic enhancement.

## PROGRAM DESCRIPTION

The stormwater runoff problems in Orange County are similar to those in other cities throughout the country. Orange County emphasizes water quality problems more than many other cities, however, because of the great number of lakes and waterways in the region and the area's dependence on tourism as a major source of income.

### Area Characteristics

The important area characteristics to consider when designing and implementing source controls are topography, climate, rainfall characteristics, hydrology, land use, and soil types. Orange County is in east central Florida with Orlando being the largest city in the region, as shown in Figure 39.

The topography of the Orange County area is generally flat with increasing elevations from east to west. The eastern portion is predominantly less than 11 m (35 ft); the western highlands, above 32 m (105 ft), are gently rolling sandy hills; the intermediate area in the center of the county is an undulating topography varying between 11 m (35 ft) and 32 m (105 ft).

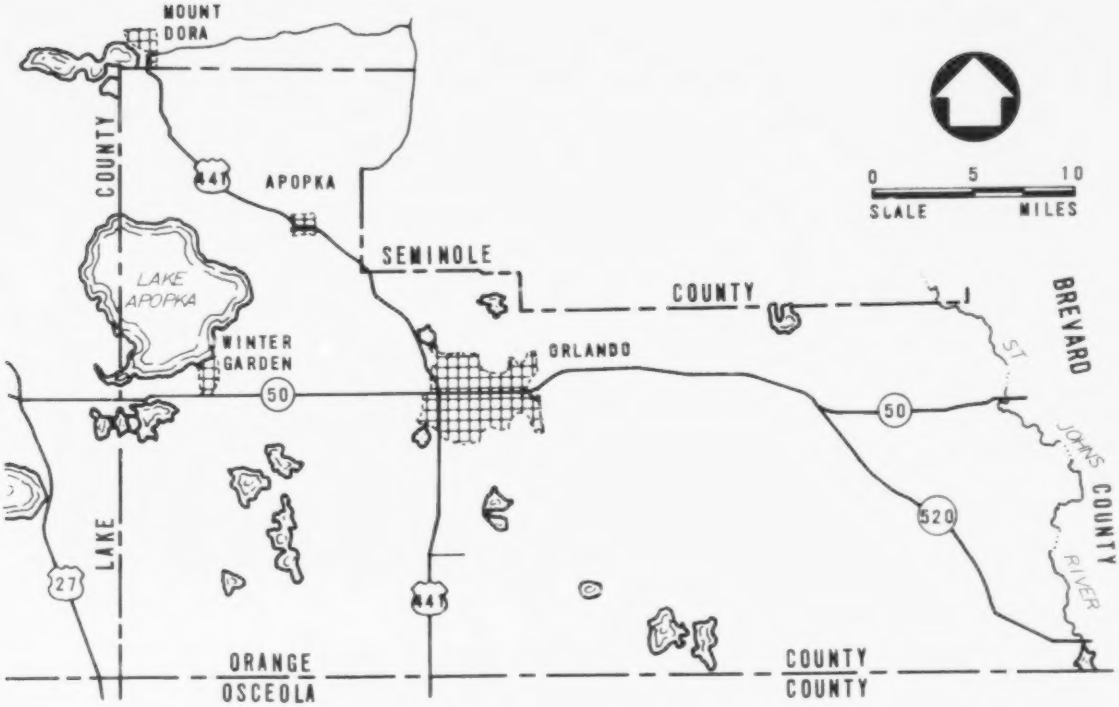


Figure 39. Orlando and Orange County, Florida [1].

The climate of Orange County is much like that of peninsular Florida, a variation between humid wet and dry seasons creating a semitropical environment. The average temperature is about 21°C (70°F).

Over 50% of the annual precipitation occurs from June through September. These rainstorms are usually afternoon showers or thunderstorms that can produce between 1.3 and 5.1 cm (0.5 and 2 in.) of precipitation in less than 1 hour. The frequency of occurrence of these storms can cause flooding.

Soil types and their capacities to infiltrate precipitation are interrelated with the area's hydrology. The western third of Orange County is designated as a prime recharge area. The soil capabilities and hydrology in Orange County are shown in Figures 40 and 41. The land use in specific areas of soil affects the natural rates of percolation.

## Problem Assessment

The environmental impacts from stormwater runoff that could adversely affect the community and receiving waters are caused by dissolved oxygen depletion, high pathogen concentrations, and increased nutrient loadings. Socioeconomic impacts to the community might include aesthetic deterioration and interference with recreational uses.

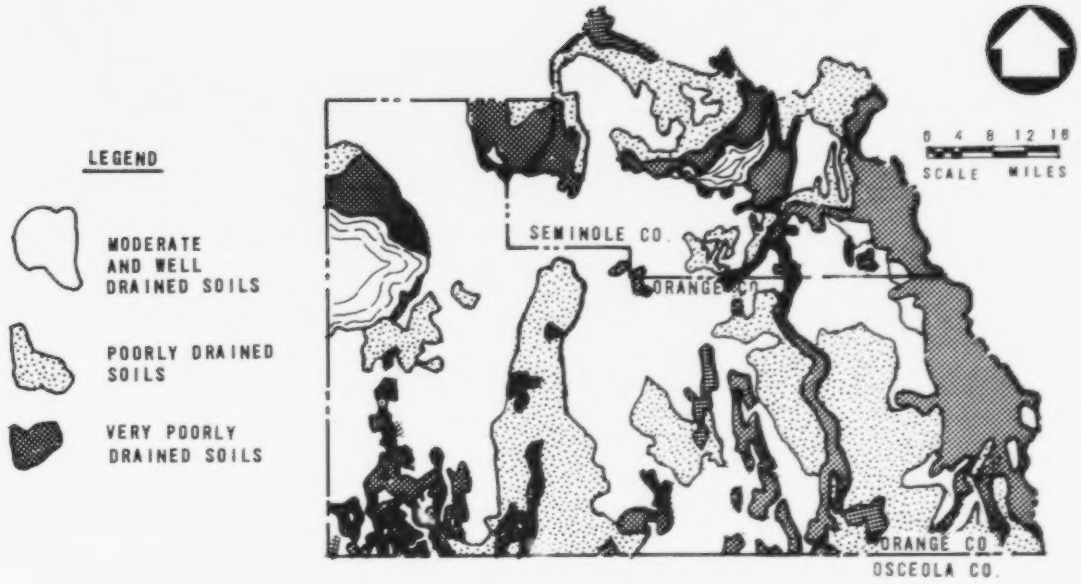


Figure 40. Soil capability of Orange County, Florida [2].

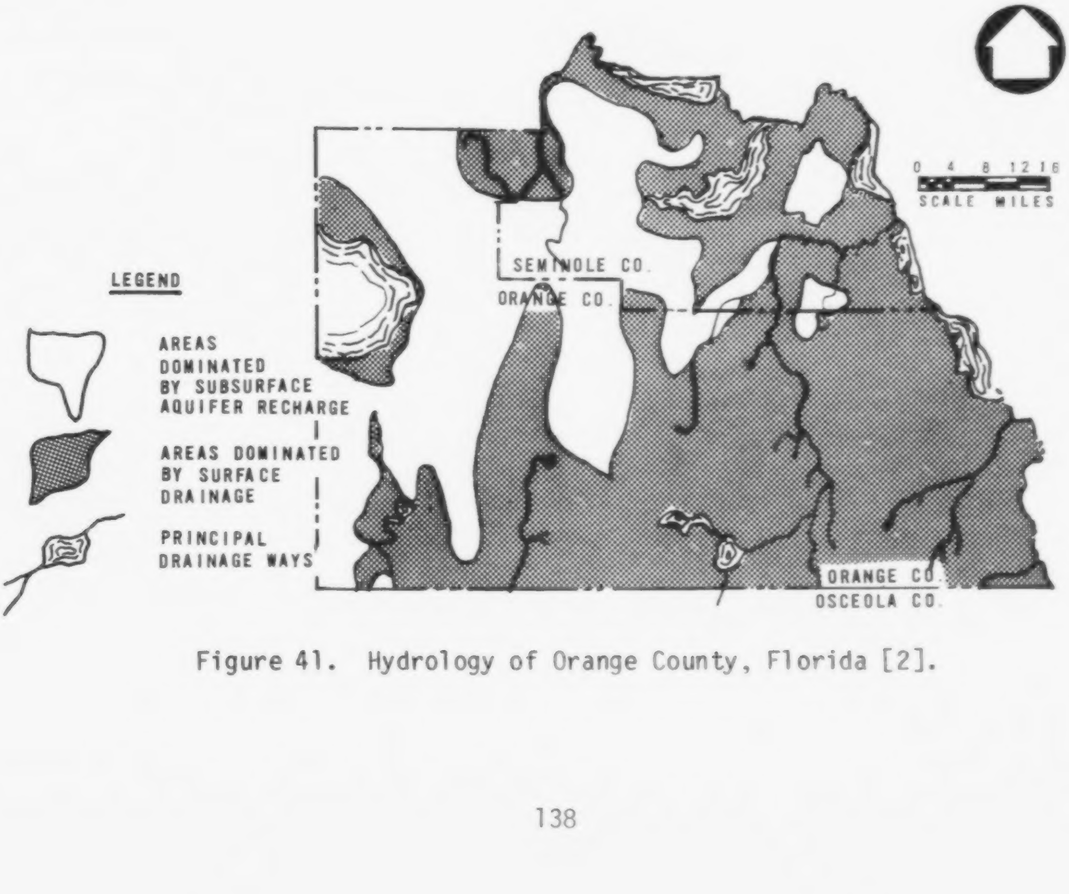


Figure 41. Hydrology of Orange County, Florida [2].

The problem in Orange County is maintaining water quality and simultaneously encouraging community growth and development. The extensive waterways and lakes in the Orange County area offer scenic beauty, recreation, and attract tourism, but the resulting development and growth can degrade the quality of the waterways and lakes.

Urban residential, urban commercial/industrial, and agricultural/improved pasture are the land uses contributing the largest pollutant loads to the receiving waters in the Orange County area. Other nonpoint sources include construction activities, hydrologic modification, saltwater intrusion, and silviculture. The nonpoint source loading rates for the major pollutants in Orange County area are compared in Table 61.

Table 61. POLLUTANT LOADING AND CONCENTRATION COMPARISON BETWEEN LAND USES AND NATURAL AREAS IN ORANGE COUNTY [3]

| | Pollutant loading, lb/acre·yr | | | | Pollutant concentration, mg/L | | | |
|---|---|---|---|---|---|---|---|---|
| Land use | $BOD_5$ | Suspended solids | Total nitrogen | Total phosphorus | $BOD_5$ | Suspended solids | Total nitrogen | Total phosphorus |
| Residential | 28.9 | 310.3 | 5.9 | 1.6 | 5.0 | 54.4 | 1.03 | 0.28 |
| Agricultural/ improved pasture | 15.1 | 391.7 | 5.6 | 1.0 | 7.0 | 180 | 2.58 | 0.46 |
| Commercial | 78.9 | 884 | 12 | 2.4 | 9.1 | 101 | 1.38 | 0.28 |
| Natural[a] | 6.7 | 31.3 | 3.3 | 0.15 | 2.7 | 8.7 | 1.35 | 0.06 |
| Well-drained soils | 0.0 | 0.0 | 0.0 | 0.0 | -- | -- | -- | -- |
| Flatwoods | 2.3 | 25.1 | 2.1 | 0.1 | -- | -- | -- | -- |
| Range | 2.3 | 25.1 | 2.1 | 0.1 | -- | -- | -- | -- |
| Swamp | 12.3 | 24.3 | 4.9 | 0.2 | -- | -- | -- | -- |

a. Weighted averages used.

Increases in the loading rates for the three major developed land uses range from 4 to 42 times the loading from natural undeveloped areas. Total phosphorus and sediments have been determined to be the pollutants with the greatest impact on the receiving waters [3].

## Countermeasure Philosophy

The BMP countermeasure strategy in Orange County has been implemented by the construction of low structural and nonstructural facilities. These controls are supported by good maintenance practices and are required by ordinance.

Orange County has several programs oriented toward solving water quality problems resulting from stormwater runoff. Southern Florida Water Management District requires the detention of the first 2.54 cm (1 in.) of rainfall runoff and allows for release at specified rates for control of floodwaters [4]. Orange County has stormwater and subdivision regulations to prevent further water quality degradation, which have also helped to control erosion, reduce flooding, and recharge the groundwater.

Orange County now requires all new development to provide the retention of the first 2.54 cm (1 in.) of rainfall, but with acceptance of the Orlando 208 study, new requirements will be mandated [4, 5]. The new regulations will be oriented toward recharge capacities, requiring the retention of the first 1.27 cm (0.5 in.) of runoff on soils with high percolation rates and the first 0.9 cm (0.35 in.) of runoff on soils with low percolation rates [4, 6].

Implemented Controls--

The abatement practices for specific land use categories were selected during the formation of the countermeasure strategy. Several examples of the abatement practices have been studied for operation and performance, costs and resources, and impacts.

Percolation is the predominant method used for treatment of stormwater runoff in Orange County. Alternative methods of percolation, corresponding to percolation rate potentials of the areas served, are first flush diversion, retention, swales, underdrains, detention, and natural treatment. Vacuum sweeping and fabric bags have also been implemented as abatement practices.

Diversion/Percolation--Because the greatest concentration of pollutants occurs in the first flush of the watershed runoff, facilities have been designed to divert and capture the first flush and allow the remainder of the water to bypass the facility to a receiving water. The diverted stormwater is then percolated in the basin, preventing the majority of the pollutants from entering the surface water. A schematic of the diversion structure and percolation pond is shown in Figure 42.

Retention/Percolation--Percolation ponds without diversion structures are also used in Orange County. A total capture percolation pond built on well-drained soils is located at the Wimbleton Apartments. The facility, shown in Figure 43, serves almost 5.77 ha (14 acres) of watershed and has a capacity of 1,974 $m^3$ (1.6 acre-ft). The pond is 100% efficient, but exceeds design requirements and has maintenance problems because of excessively steep side slopes [7].

Swale/Percolation--Swale/percolation facilities are also an effective method for reducing the major runoff pollutants in Orange County. A swale/percolation facility between an apartment complex and a lake is shown in Figure 44. This facility has an inlet structure from the service area and overflows to the lake through an overflow structure if the capacity of the facility is exceeded. Stormwater remains in the swales until percolated.

Underdrains--Underdrains in residential areas have shown relatively high efficiency in reducing the pollutant loading from stormwater runoff, but also are the most expensive to implement. Underdrains use percolation as the method of treatment and consist of perforated pipe enveloped in gravel to provide drainage beneath swales, as shown in Figure 45. The effluent is then discharged into a storm sewer or a receiving water. Design recommendations for high efficiency include a minimum of 0.3 m (1 ft) of soil between the surface and the top of the gravel and the use of some permeable layer, such as straw, between the soil and gravel to prevent soil from washing into the

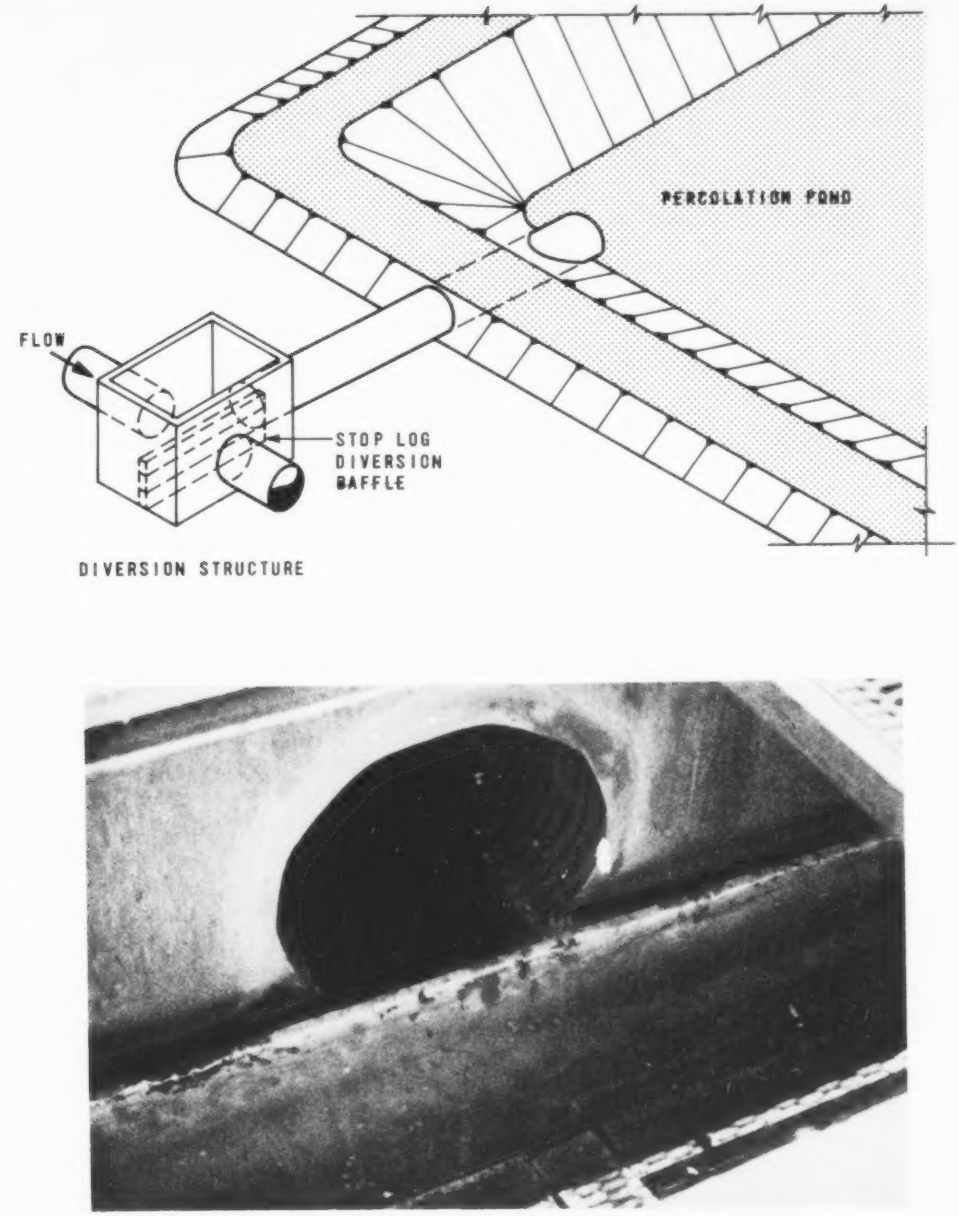


Figure 42. Diversion/percolation pond at 8 Days Inn: (a) schematic of diversion/percolation facility, (b) diversion structure, showing stop log diversion baffle.

voids. Underdrains not only operate effectively, but provide an aesthetically pleasing solution with virtually no maintenance requirements. An underdrain cross-section and a typical surface view are shown in Figure 45.

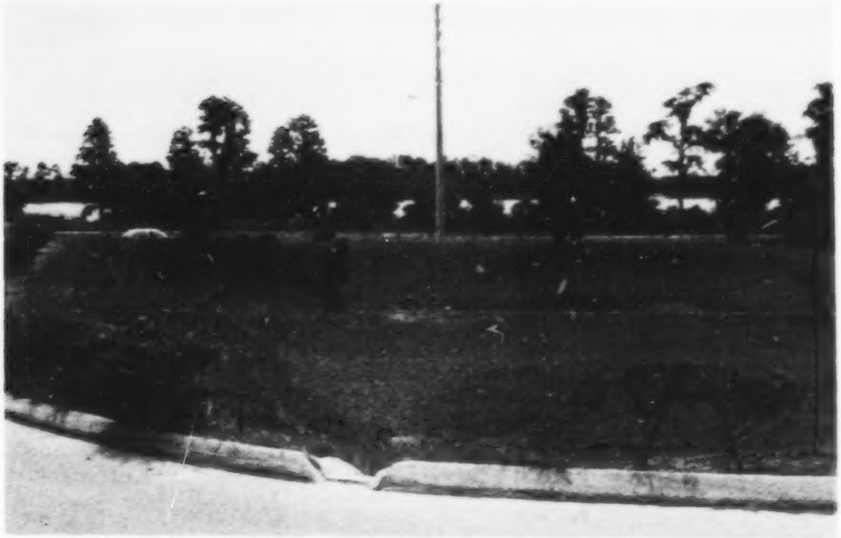


Figure 43. Total capture percolation facility.

Residential Swales--Percolation of stormwater in swales (grassed depressions) has provided efficient treatment when located on soils with high percolation rates. Swales have exhibited low performance when located on poorly drained soils or soils excessively compacted during construction. Problems encountered with swales include erosion on steep side slopes and insect breeding associated with standing water.

Detention/Sedimentation--The sedimentation basin at Prairie Lake provides relatively good pollutant removal rates for the unit cost per impervious area serviced. The facility is 5.5 by 11.5 m (18 by 38 ft) and the depth tapers from 1.8 m (6 ft) at the inlet to 0.15 m (0.5 ft) at the outlet [7]. A chickenwire fence at the inlet traps large debris, and a baffle extends to a depth of 0.6 m (2 ft) into the middle of the pond and acts as an energy dissipator to prevent short-circuiting. The facility has experienced resuspension of sediments deposited from previous storms.

Natural Treatment--Natural treatment using a cypress stand provides one of the least expensive treatment methods for stormwater runoff and yet its reduction of pollutant loadings is among the highest. Cypress stands are characterized by marsh vegetation, small cypress at the exterior, and mature cypress with intermittent ponds and sloughs in the interior. Slow-moving flow through the stand allows vegetation to grow in the ponds and peat to accumulate. The erosion rates are also low. Stormwater is channeled into the cypress stand

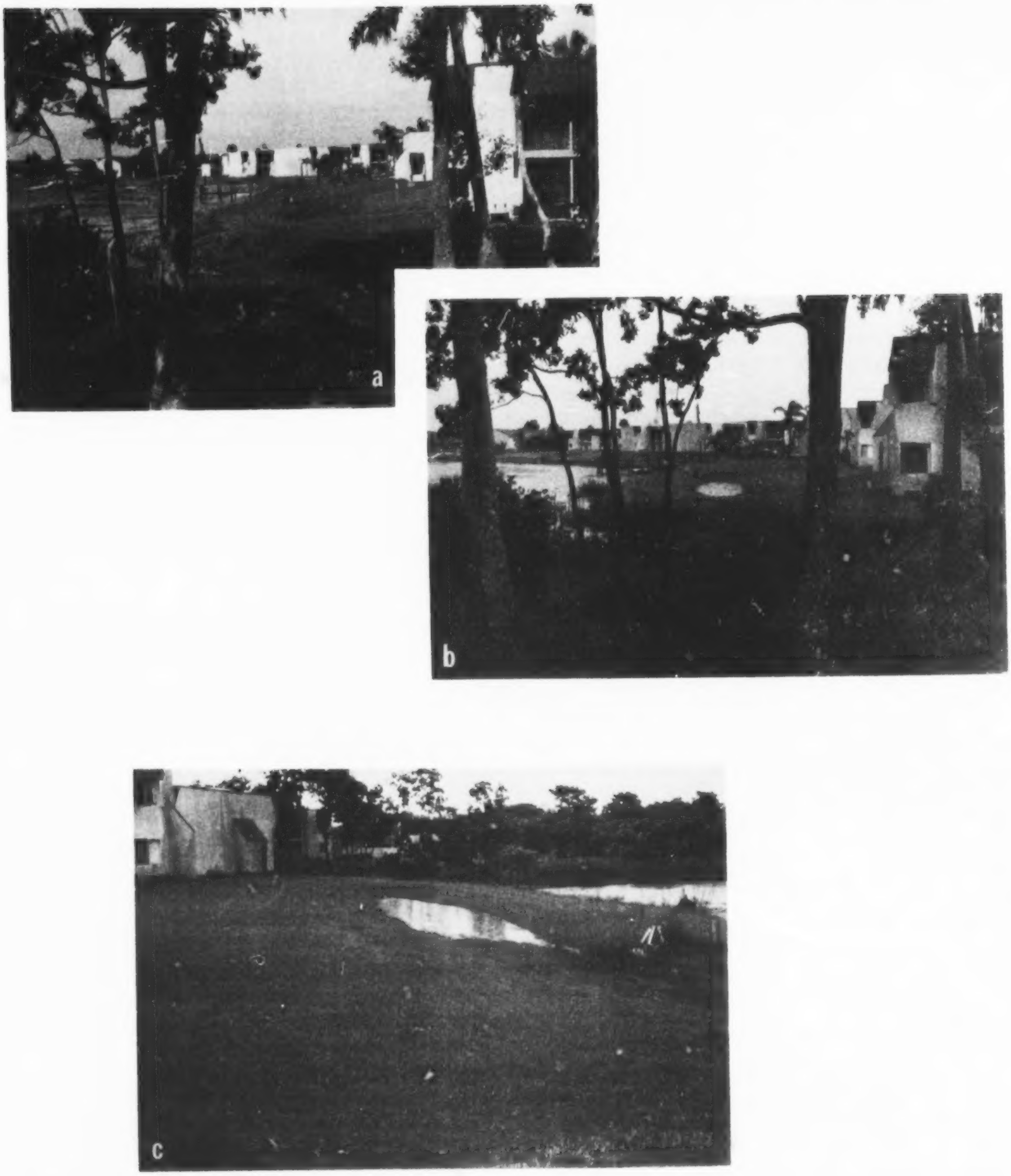


Figure 44. Swale/percolation: (a) dry swales before rain; (b) swales during rain, (c) facility filling during rain, with overflow structure at the right.

with the outflow a function of the inflow rate, storage capacity, vegetation density, and infiltration rate.

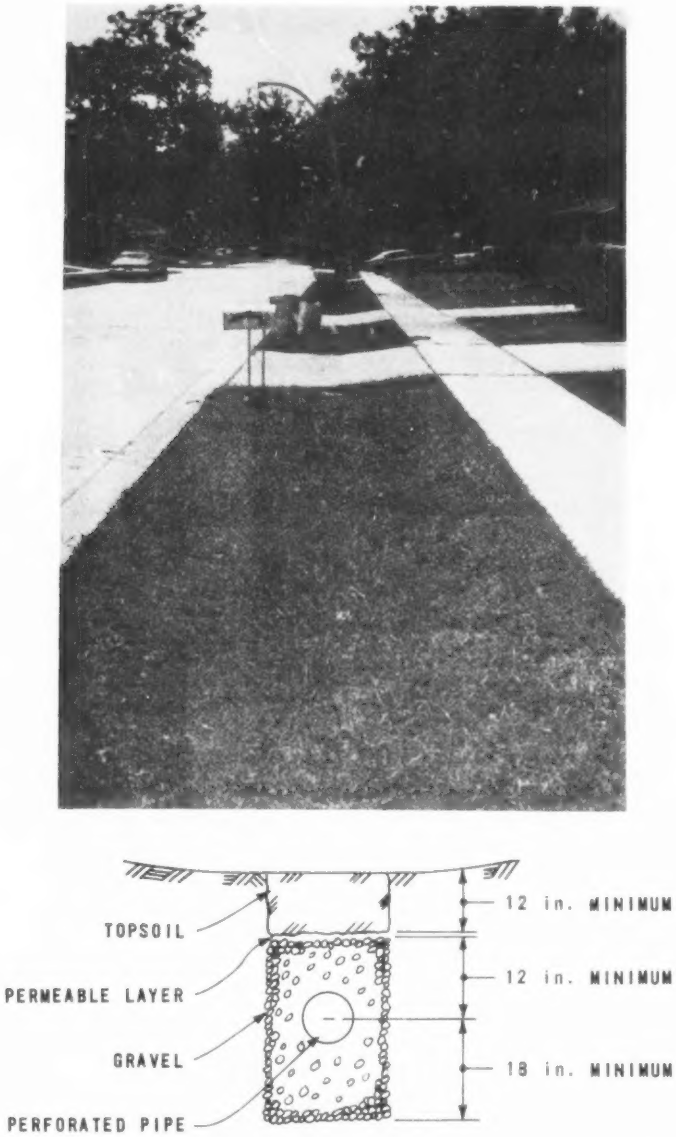


Figure 45. Residential underdrain: (a) typical surface view of swale above underdrain, and (b) schematic of underdrain system [7].

Fabric Bags--Fabric bags have been mounted in inlets to stormdrains to filter the stormwater. Although this method is the least expensive, it required high maintenance and had a lower operational efficiency for suspended solids than did other methods. The effectiveness on organics removal was high, but this method was not used after the study period.

Vacuum Sweeping--Vacuum sweeping was implemented as an operation and maintenance measure to reduce the pollutant loading of runoff from a shopping mall parking lot. The parking lot area required daily vacuum sweeping of about 2,977 m (9,760 ft) of the internal curb and weekly sweeping of approximately 2,657 m (8,710 ft) of the external curb [7]. The vacuum sweeping efficiency approached 80% for suspended solids in this application. The vacuum sweeping study area is shown in Figure 46.

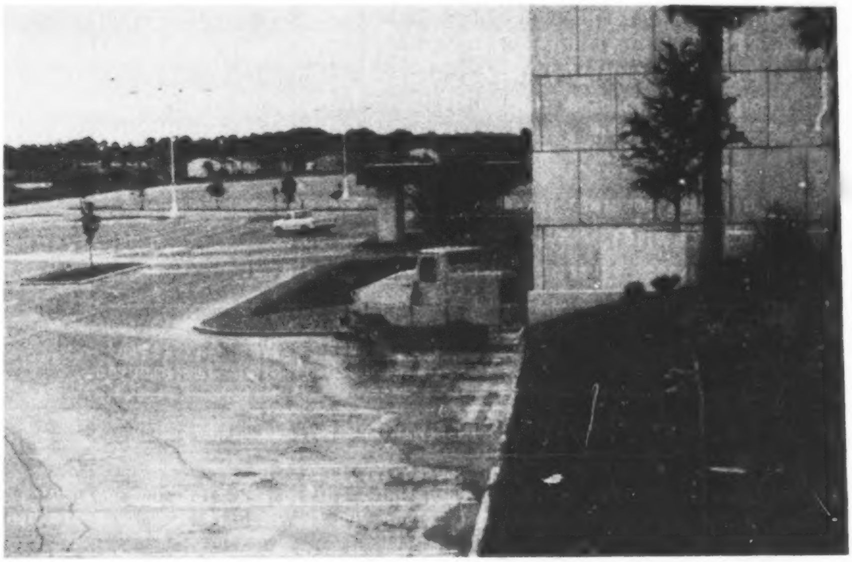


Figure 46. Vacuum sweeping study area at Altamonte Mall.

Design Criteria--

The design criteria for stormwater runoff controls should use information on the permeability of the ground surface, the type of vegetation, the area of drainage basin, the soil characteristics, and the pollutant loading rates. These factors were used in a manual entitled "Stormwater Management Practices Manual" that provides guidance in stormwater facility design in Florida [8]. The approach assumes worst case conditions where 100% of the watershed contributes to runoff.

The SCS soil groupings for runoff potential and runoff curve numbers for selected land uses corresponding to the soil groupings are the basis for evaluating runoff from a drainage area. Soil groupings classify soils from lowest to highest runoff potential: soil Group A, lowest runoff potential; B, moderately low runoff potential; C, moderately high runoff potential; and D, highest runoff potential.

For each of the soil group classifications, runoff curve numbers (CN) are determined for various land use types and conditions of vegetative cover. These numbers range from 0 to 100, with 100 representing the most impervious soil condition producing the highest runoff [8]. Curve numbers for a specific land use and soil group may be further adjusted to reflect soil moisture conditions. As soils become saturated, they become more impervious.

These design considerations, which describe the physical conditions of the site, are used with regulatory design limits prescribing the volume of runoff to be diverted to a detention/percolation facility (including possible first flush loadings). As an example of the use of the design guidelines developed for the Florida area, a comparison is made between a constructed basin and a basin designed by the methods presented in the manual described previously.

## EXAMPLE DESIGN OF A STORMWATER DIVERSION/PERCOLATION FACILITY

Determine the pounds of pollutants removed per year, the size of the facility required, the capital cost, and the present value for a diversion/percolation facility from the information given below and by using Figure 47.

### Specified Conditions

1. Total area served = 4.6 acres
2. Impervious area served = 3.8 acres
3. Runoff to be diverted for treatment = 1.15 in.
4. Type A soils in percolation basin.

### Assumptions

1. The soil is saturated and therefore responds as impervious soil (CN = 100).
2. Pollutant loadings in lb/acre·yr are

   Suspended solids = 254
   $BOD_5$ = 36.5
   Total nitrogen = 7.8
   Total phosphorus = 1.1

### Solution

1. Determine the pounds of pollutants removed per year:
   a. Locate the total area served on the "contributing watershed area" axis.
   b. Project a vertical line to the 1 in. diversion line (maximum value of the graph)
   c. Read pounds of pollutants removed per year:

      Suspended solids = 1,500
      $BOD_5$ = 168
      Total nitrogen = 50
      Total phosphorus = 5.5
2. Determine the percolation basin size:
   a. From the intersection of the 1 in. diversion line and the vertically constructed line, draw a horizontal line to the pivot line for type "A" basin soils in the top left-hand quadrant.

      Note that the CN for impervious areas = 100
   b. From the intersection of the type "A" basin soils pivot line, construct a vertical line to the CN = 100 line in the lower left-hand quadrant.
   c. From this point, draw a horizontal line through the basin volume axis to the capital and present value graphs in the lower right-hand quadrant.

      Reading from the basin volume axis, the required percolation pond size is approximately 0.65 acre-ft.

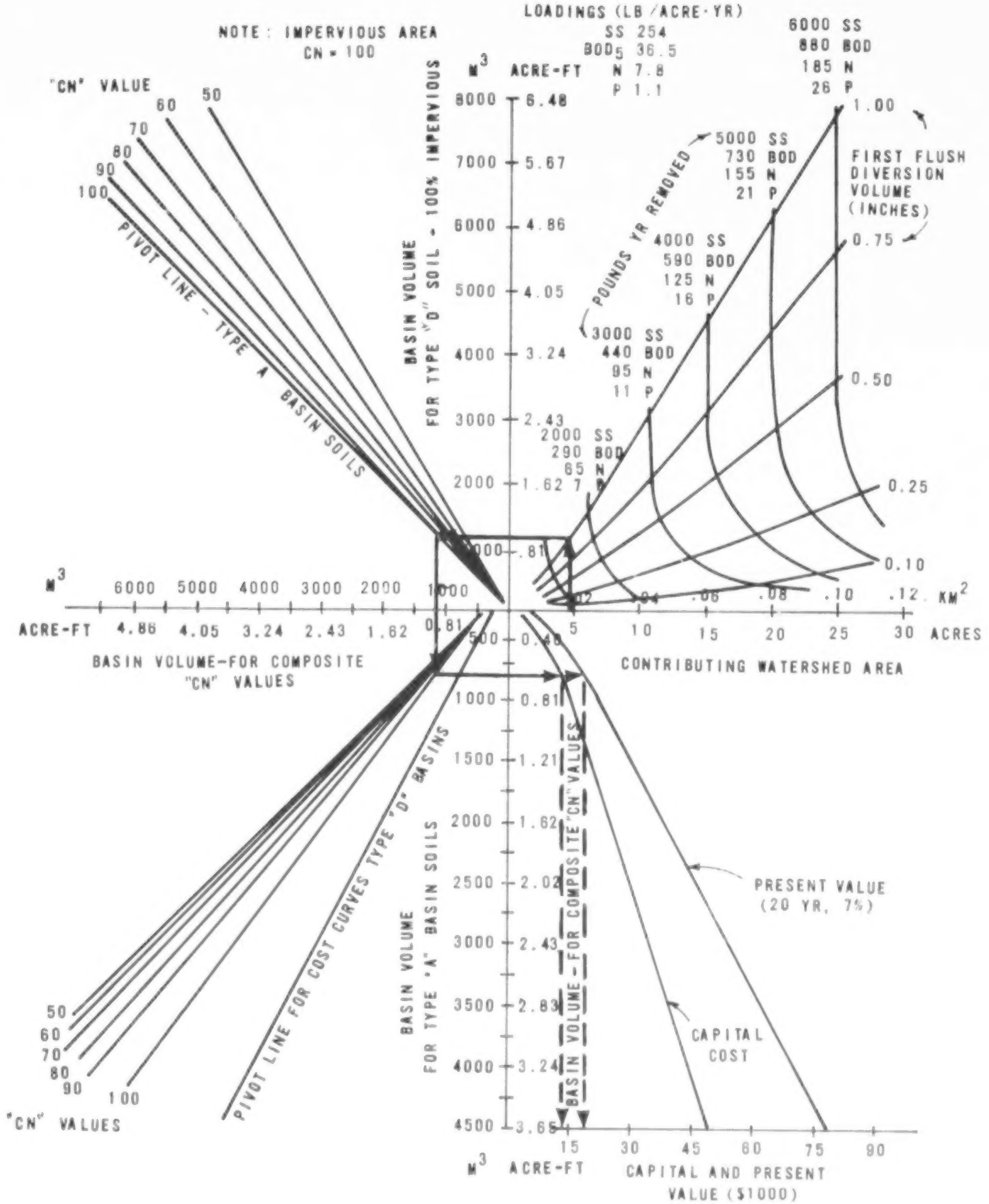


Figure 47. Size, efficiencies, and cost of diversion/percolation basins (ENR 3000) [8].

3. Determine the capital and present value of the diversion/percolation facility.

From the intercepted points on the capital and present value lines, draw a vertical line to the base axis of the right-hand quadrant

Capital cost = $14,067
Present worth = $20,336

Comment

The use of this graph is limited to Florida as data used regarding soil, watershed, and meteorology apply specifically to Florida.

A comparison of values from an implemented facility and the results from this example is shown in Table 62. Although the design graph is only an aid for first-cut design, it shows good correlation between values of an implemented facility.

Table 62. DESIGN AND IMPLEMENTED FACILITY COMPARISON[a]

| | Graph projected values, using Figure 9 | Implemented facility, 8 Days Inn |
|---|---|---|
| Area served, acres | 4.6 | 4.6 |
| Facility size, acre-ft | 0.65 | 0.44 |
| Pollutant loading, lb/acre·yr | | |
| BOD5 | 36.5 | 46.9 |
| Suspended solids | 254 | 356 |
| Total nitrogen | 7.8 | 10.57 |
| Total phosphorus | 1.1 | 1.92 |
| Pollutant removal, lb/yr | | |
| BOD5 | 168 | 214 |
| Suspended solids | 1,500 | 1,622 |
| Total nitrogen | 50 | 48 |
| Total phosphorus | 5.5 | 8.7 |
| Capital cost, $ | 14,067 | 14,380 |
| Present worth (20 yr at 7%), $ | 20,336 | 26,400 |

a. ENR 3000.

## OPERATION AND PERFORMANCE

The operation and performance of Orange County's BMPs have provided data on unit effectiveness of low structural and nonstructural facilities. The analysis of the data has identified design criteria, efficiencies, operation needs, and associated maintenance problems.

### Operation

The stormwater facilities in Orange County have low manpower requirements. Most facilities have been designed for specific loadings or as self-activating units and therefore, require minimal operational control. Facility operation may involve changing the elevation of baffles in diversion chambers to capture different amounts of the first flush or controlling the release of flow from detention/sedimentation ponds. Self-activating stormwater controls are shown in Figure 48.

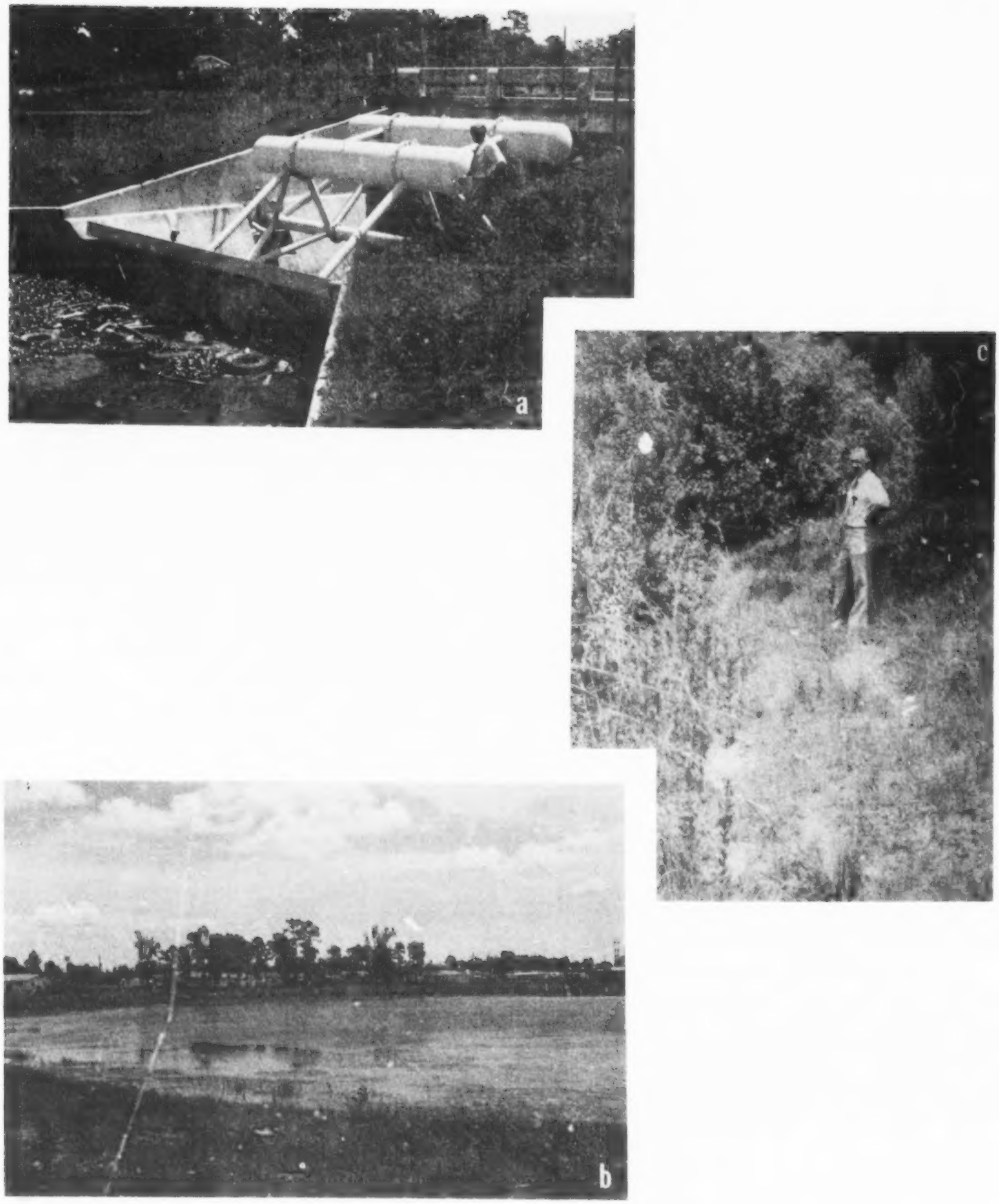


Figure 48. Self-activating stormwater controls: (a) regional self-activating flood gate, (b) regional stormwater percolation/evaporation pond, (c) small self-activating diversion/percolation pond.

## Maintenance

Maintenance of low structural or nonstructural stormwater facilities or practices include:

- Debris removal
- Landscape care
- Vector and insect control
- Aquatic vegetation control
- Removal or scarification of sediment deposits
- Street sweeper maintenance
- Fabric bag maintenance
- Repair of erosion damage

Generally, the facilities serving urban commercial/industrial or residential areas will contribute larger quantities of debris, solids, pollutants, and substantially larger quantities of runoff because of the greater areas of impervious cover.

Multiuse facilities usually provide, in addition to their designated purpose, aesthetic appeal or recreation-oriented use and require higher maintenance. Facilities in public areas will most likely require maintenance equal to that of a multiuse facility. If there is public access to the facility, a higher degree of landscape manicuring, control of litter, and continual care will be required.

Stormwater facilities that have a fill/drain sequence are likely to need more maintenance than facilities that remain full of water. Detention ponds or sedimentation ponds, for example, would collect debris and solids that would have to be removed during dry periods. These facilities may have short periods of minimal ponding, when insects might breed and odors develop. All of these problems would increase maintenance requirements.

Orange County requires developers not only to implement stormwater runoff controls, but to maintain them. In most cases, developers have assumed maintenance responsibilities for the stormwater facilities, but in the cases where they do not, Orange County will maintain the facility and levy a fee against the development for the required work. For regional flood control and stormwater runoff facilities, maintenance is the responsibility of Orange County, the City of Orlando or, in the case of facilities built in conjunction with state highway projects, maintenance is the responsiblity of the State Department of Transportation [4].

## Performance

The unit performance of the Orange County BMPs has shown substantial success in implementing stormwater runoff controls that will help meet water quality goals. The aggregate effect of these BMPs has not been determined, although the cumulative results of several control measures are expected to be substantial.

Suspended solids, $BOD_5$, total nitrogen, and total phosphorus have been identified as the main pollutant loads that should be reduced. Percolation facilities on soils with high percolation rates have shown exceptional pollutant removal. The percolation basins at Wimbleton Apartments and the diversion/percolation basin at 8 Days Inn are nearly 100% effective in removing major pollutants [7]. Although these facilities serve relatively small areas with varying amounts of impervious land, the efficiencies indicate the potential value of implementing these types of facilities for control of stormwater runoff in entire basins.

Facilities that have performed well, but not with total efficiency, include natural treatment via cypress stands, underdrains, swales/percolation, and sedimentation basins. Vacuum sweeping was of considerable value in eliminating suspended solids from a parking lot, but had less efficiency on other pollutants. Similarly, the fabric bag was effective for reducing $BOD_5$ and total nitrogen, but was of marginal value in other pollutant reductions. The effectiveness of the BMPs used in Orange County is summarized in Table 63.

The reduced volume of stormwater runoff on a basinwide basis has not been determined. Substantial reduction of runoff in some areas has been noted on a qualitative basis by maintenance crews [4]. The reduction of runoff flows and their subsequent percolation into the groundwater may be considered a secondary benefit of BMP controls in this area.

## COST AND RESOURCES

The costs and resources associated with the BMPs implemented in Orange County include capital and annual operation and maintenance. The construction of the stormwater facilities is the main expense. The maintenance costs and resource use are minimal.

### Capital Costs

The capital costs of stormwater control facilities for new development are the responsibility of the developer. Larger regional facilities constructed by the county are financed by the county general fund or by the State Department of Transportation. This state agency works with the counties to provide stormwater facilities with the construction of new highways.

Capital costs for several types of facilities are given in Table 64. Unit costs are calculated for impervious and total area served by each facility, excluding the cost of land.

The ratio of impervious land to total land served by each control measure varies significantly in some cases; therefore, the unit costs may be compared more easily on an impervious area basis.

### Operation and Maintenance Costs

Annual costs to operate and maintain stormwater facilities vary with the type of practice implemented. BMPs with significantly lower operation and maintenance costs are maintained by private individuals or homeowners, as in

## Table 63. PERFORMANCE OF BMPs IN ORANGE COUNTY [3]

| Stormwater control measure | Location | Storms sampled | Influent load, lb/yr | | | | Effluent load, lb/yr | | | | Avg efficiency, % | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | $BOD_5$ | N | P | SS | $BOD_5$ | N | P | SS | $BOD_5$ | N | P | SS |
| Diversion/ percolation | 8 Days Inn | 6 | 213.81 | 48.1 | 8.74 | 1,624.00 | 0.149 | 0.05 | 0.05 | 1.98 | 99+ | 99+ | 99+ | 99+ |
| Retention/ percolation | Wimbledon Apartments | 6 | 433.14 | 46.76 | 14.70 | 3,035.56 | 0 | 0 | 0 | 0 | 100 | 100 | 100 | 100 |
| Swale/ percolation | Shoals Apartments | 4 | 221.95 | 72.42 | 21.36 | 6,092.98 | 16.36 | 9.44 | 1.72 | 237.91 | 93 | 90 | 92 | 96 |
| Underdrain[a] | Winter Park | 4 | 260.10 | 53.10 | 14.40 | 11,477.00 | 11.70 | 24.30 | 1.08 | 138.6 | 96 | 54 | 93 | 99 |
| Residential swale[b] | Tuskawilla | 2 | 75.14 | 15.34 | 4.16 | 3,315.00 | 46.67 | 12.85 | 4.40 | 362.47 | 38 | 16 | 0 | 89 |
| Detention/ sedimentation | Prairie Lake | 6 | 424.94 | 130.31 | 47.98 | 17,884.39 | 171.21 | 31.38 | 11.55 | 2,936.54 | 60 | 76 | 76 | 84 |
| Cypress stand | Florida Technological University | 8 | 319.21 | 99.44 | 17.63 | 49,784.31 | 34.89 | 5.18 | 0.48 | 274.05 | 89 | 95 | 97 | 99 |
| Fabric bags | Lake Sylvan | 9 | 440.36 | 94.44 | 19.70 | 1,566.01 | 88.38 | 40.91 | 16.16 | 1,228.16 | 80 | 57 | 18 | 22 |
| Vacuum sweeping[c] | Altamonte Mall | 6 | 1,474.31 | 151.77 | 28.91 | 61,140.00 | 1,445.40 | 144.54 | 28.91 | 10,119.80 | 2 | 5 | 0 | 83 |

a. Influent loading estimated from average values at the Shoals Apartments. The volume of surface runoff was assumed equal to the volume of underdrained water.

b. Influent loading estimated from average values at the Shoals Apartments without swales. The volume of surface runoff was considered equal for comparison purposes.

c. Influent load estimated from parking lot discharge sampling plus the loads collected by the cleaning equipment. No control areas were used to check the significance of the data.

the case of underdrained swales. A comparison of the operation and maintenance costs for BMPs in Orange County is shown in Table 65.

Table 64. CAPITAL COSTS OF BMPs IN ORANGE COUNTY[a,b] [7]

| Stormwater control measure | Total area serviced, acres | Impervious area serviced, % | Total capital costs, $ | Unit capital costs, $/acre | Impervious area unit costs, $/acre |
|---|---|---|---|---|---|
| Diversion/ percolation | 4.56 | 84 | 14,380 | 3,154 | 3,754 |
| Retention/ percolation | 13.79 | 26 | 19,654 | 1,425 | 5,482 |
| Swale/ percolation | 4.92 | 74 | 14,556 | 2,959 | 3,998 |
| Underdrains | 9.0 | 20 | 23,233 | 2,581 | 12,908 |
| Residential swale | 2.6 | 48 | 4,949 | 1,904 | 3,966 |
| Detention/ sedimentation | 4.4 | 80 | 3,637 | 827 | 1,033 |
| Cypress stand | 24.1 | 67 | 3,823 | 159 | 237 |
| Fabric bags[c] | 4.8 | 49 | 176 | 37 | 75 |
| Vacuum sweeping[d] | 43.5 | 100 | 56,267 | 1,292 | 1,292 |

a. ENR 3000.

b. Includes 15% for engineering and legal; land costs are not included.

c. Fabric bags must be replaced every 2 years; two fabric bags are installed.

d. Assumes cleaning equipment is used exclusively for the area being swept.

Table 65. BMP OPERATION AND MAINTENANCE COSTS IN ORANGE COUNTY[a]

| Stormwater control measure | O&M costs, $/acre·yr |
|---|---|
| Diversion/percolation | 294 |
| Retention/percolation | 642 |
| Swale/percolation | 459 |
| Underdrains | 0 |
| Residential swale | 73 |
| Detention/sedimentation | 532 |
| Cypress stand | 28 |
| Fabric bags[b] | 239 |
| Vacuum sweeping[c] | 261 |

a. ENR 3000.

b. Fabric bags must be replaced every 2 years; two fabric bags are installed.

c. Labor and equipment upkeep.

## Cost Effectiveness

A comparison of the cost effectiveness of the BMPS in Orange County is presented in Figure 49.

The efficiency of facilities ranges from 0 to 100% removal of major pollutants. The capital costs vary from $30 to $5,228/impervious hectare ($75 to $12,908/impervious acre). The cost effectiveness of removing specific pollutants from stormwater by the implemented controls is shown in Table 66.

The capital costs of percolation basins at various levels of efficiency and soil percolation rates are shown in Figure 50. Soil percolation rates have a major influence on the cost of facilities for higher removal efficiencies. Economies of scale are greatly inflated by a factor of 40, between 70 and 90% removal efficiencies, for facilities on poorly drained soils.

## IMPACTS

Expanding population and urbanization have resulted in general environmental degradation. The Orlando Metropolitan 208 study has identified its major problem as general degradation of lakes and waterways because of excessive nutrient and sediment loadings. Adverse environmental impacts associated with the poor water quality of the region include [3]:

- Reduction in species habitat, type, and population
- Increase in pest species (i.e., aquatic growth, insects)
- Increase in eutrophication rates
- Adverse effects to potable water supply
- Decrease in aesthetic appeal

The countermeasure strategy is not fully implemented, but projections have been made as to the expected environmental and socioeconomic impacts resulting from full implementation.

### Environmental Impacts

The abatement strategy goal is to reduce the receiving water pollutant loading rates to natural (predevelopment) conditions. The impacts on the climate, topography, soils, and subsurface geology are anticipated to be small.

The major environmental impacts will be associated with surface and groundwater hydrology. Positive impacts are anticipated for surface water in accordance with planned strategy. These positive impacts will be long-term improvements in surface water quality, predominantly from the reduction of nutrients, solids, pesticides and herbicides, heavy metals, and increased dissolved oxygen. The elimination of pollutants from surface waters via percolation methods can potentially deposit pollutants in the groundwater.

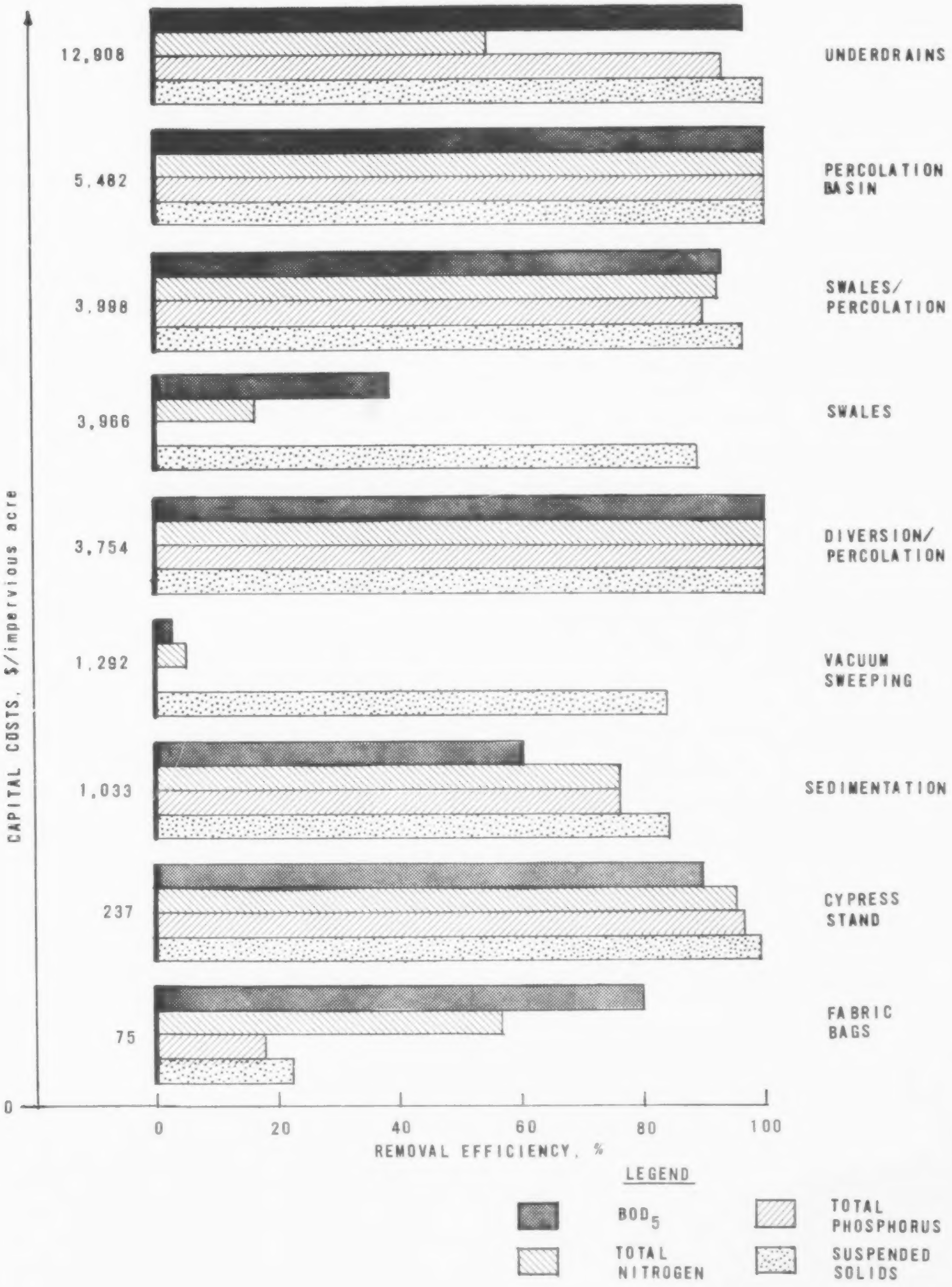


Figure 49. Cost effectiveness of BMPs in Orange County.

Table 66. COMPARISON OF COSTS FOR REMOVING $BOD_5$ AND SUSPENDED SOLIDS IN ORANGE COUNTY

| Stormwater control measure | Cost of pollutant removal, $/lb·impervious acre·yr | |
|---|---|---|
| | SS | $BOD_5$ |
| Diversion/percolation | 2.31 | 17.57 |
| Retention/percolation | 1.80 | 12.66 |
| Swale/percolation | 0.68 | 19.45 |
| Underdrains | 1.14 | 51.96 |
| Residential swale | 1.34 | 139.30 |
| Detention/sedimentation | 0.07 | 4.07 |
| Cypress stand | 0.005 | 0.83 |
| Fabric bags | 0.22 | 0.21 |
| Vacuum sweeping | 0.03 | 44.73 |

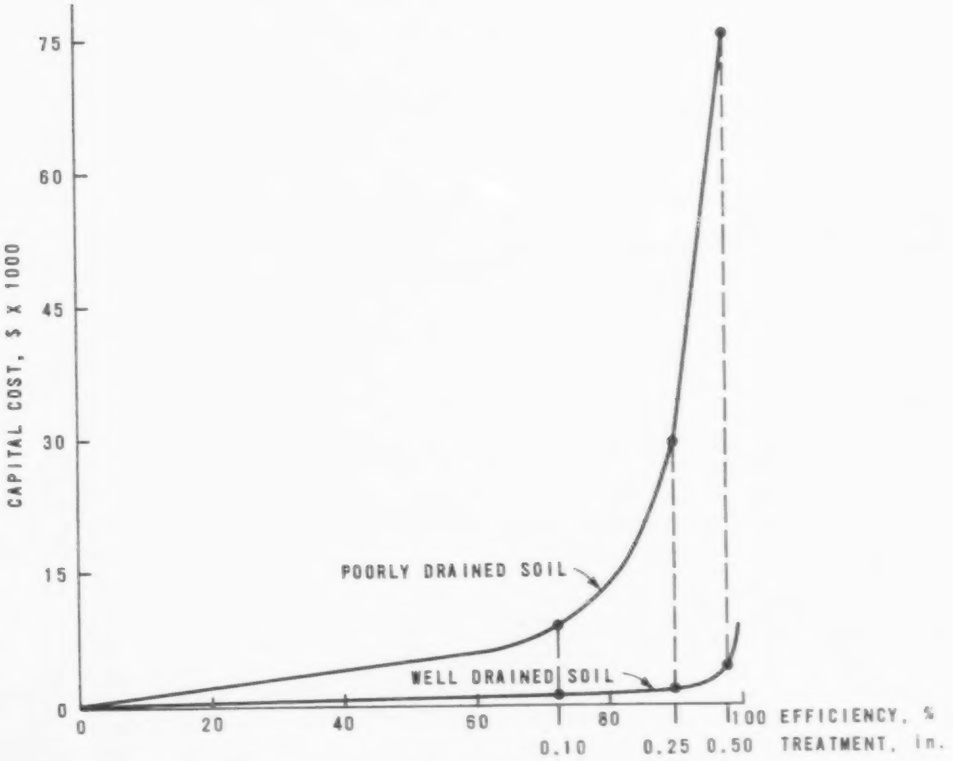


Figure 50. Cost comparison of percolation ponds on well drained and poorly drained soils [7].

The severity of the problem has not been determined; however, potential benefits to the groundwater could be derived from increased recharge rates.

It is assumed that an increase in the quality of aquatic and terrestrial habitats will also occur. Only minimal long-term positive impacts are anticipated to rare and endangered species.

Environmentally sensitive areas, such as wetlands, marshes, flood plains, forests, and timber resources, or ecologically unique areas, should not be affected by the proposed strategies.

Impacts on land uses, both existing and future, are considered to be negligible, as are impacts to air and noise pollution. The implementation of the proposed abatement strategies will not affect present land uses, and any techniques used will be integrated with existing land uses. Although future developments will be required to implement BMPs, it is not anticipated that this requirement will be a deterrent to land development or affect land uses.

## Socioeconomic Impacts

The effects of the program on population, economic activity, public acceptance, aesthetics, and future facilities are some of the considerations that must be made.

The implementation of the proposed strategies are not anticipated to have any direct effect on the population or the distribution of the population, but some demographic changes may occur if the lake degradation continues.

Public acceptance and aesthetics of program implementation are interrelated. The public's unhappiness over poor aesthetics and general water resource degradation in Orange County was the impetus for major strides in increased pollution abatement. The general public not only accepted the control of stormwaters, but supported program implementation. However, not all goals set in the strategy were totally accepted by some portions of the community; the limitation of new development in flood prone areas is an example.

Aesthetic appeal has been incorporated in many stormwater facilities. Examples of multipurpose and aesthetically pleasing facilities are a percolation basin used as a volleyball court and an apartment clubhouse modeled after a showboat, located on a detention pond, shown in Figure 51.

Several adverse economic impacts from planned strategies are anticipated in the long-term analysis. Increased costs to private industry and services for construction of stormwater control facilities will be passed on in prices to customers. Other costs to the community that might be considered excessive include the duplication of taxes associated with overlapping government authorities that regulate stormwater.

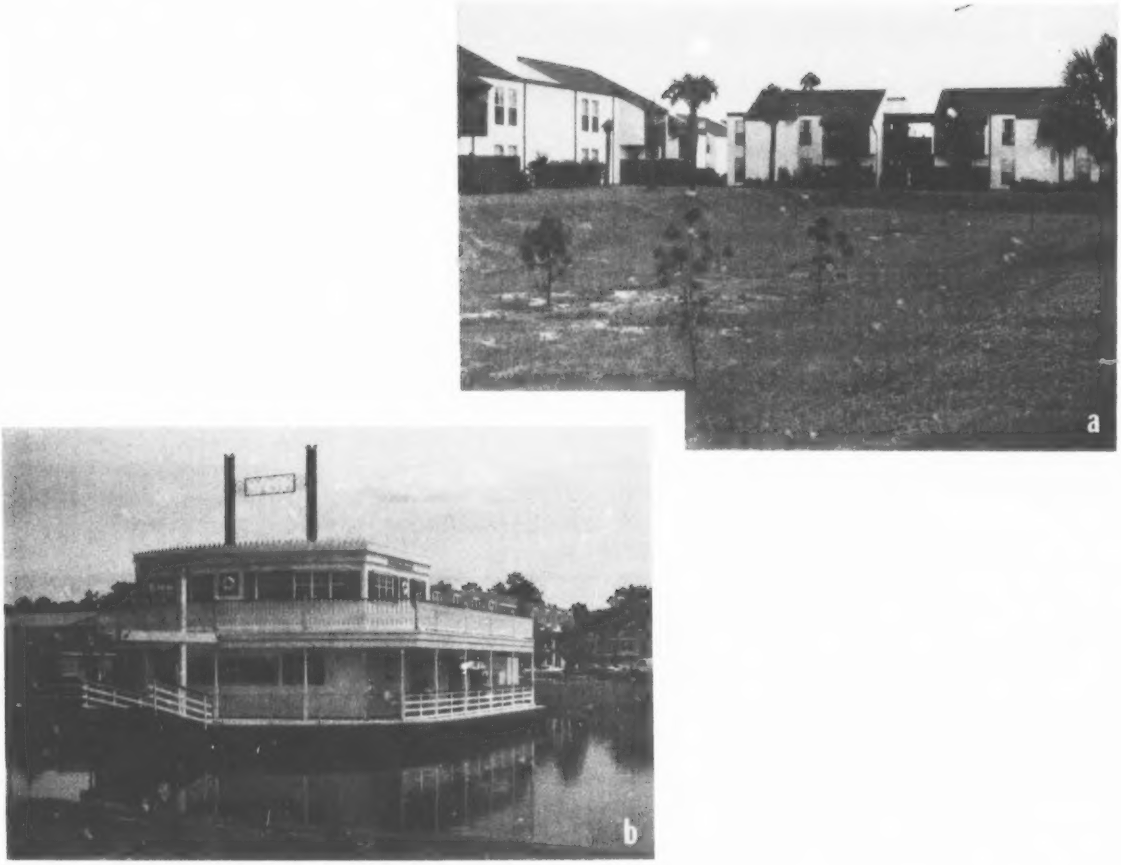


Figure 51. Multipurpose stormwater facilities in Orange County: (a) percolation basin/grassed volleyball court, and (b) detention pond/apartment complex clubhouse.

# SECTION 9

# IMPROVED STREET CLEANING PRACTICES
# SAN JOSE, CALIFORNIA

In 1976, the City of San Jose, California, undertook a comprehensive, 2 year project to demonstrate pollution abatement through improved street cleaning practices [1]. The project findings led to a recommended approach for designing street cleaning programs, which should be applicable to most areas of the country.

Although street cleaning has long been widely practiced, its major goals were litter and dirt removal and dust control (street appearance). Its use as a BMP to control stormwater pollution has not been practiced. Past studies of the effectiveness of street cleaning as a water quality control measure were very limited in scope based only on idealized strip test conditions.

The San Jose study is unique because it measured the effectiveness of street cleaning over large areas for 1 year, under many "real world" conditions. The sources of urban runoff pollutants and the wash-off effects of various-sized storms were identified, and the cost effectiveness and impacts of several types of street cleaning equipment were also evaluated. More than 20,000 samples were collected and analyzed.

A parallel study of Coyote Creek, which receives stormwater from the studied areas of San Jose, was also conducted [2]. This study provided much needed information on the effects of nonpoint urban stormwater discharges in receiving water.

## DEMONSTRATION PROJECT

The San Jose demonstration project was a comprehensive, detailed study that evaluated the performance and costs of street cleaning equipment; street surface pollutants, and their movements with wind, traffic, and rain; the bearing that such information should have on street cleaning equipment selection and program design; and environmental and some socioeconomic impacts.

### Area Characteristics

The area characteristics that affect the need for, and the methods of, street cleaning include (in generally decreasing order of importance):
(1) conditions of street surfaces, (2) traffic congestion, (3) construction projects, (4) presence of litter control programs, (5) climate, (6) type and condition of curbs, (7) presence and maturity of vegetation, (8) demolition work, and (9) unpaved driveways, roads, and parking lots [3].

Three study areas, considered representative of the variety of conditions found in many cities, were selected within the City of San Jose. These were the Tropicana (residential) area, the Keyes Street (residential/commercial) area, and a downtown (commercial) area. The locations of San Jose and the three study areas are shown in Figure 52. The areas selected represented various land uses, economic conditions, and traffic conditions, and met other criteria necessary for the demonstration project [1].

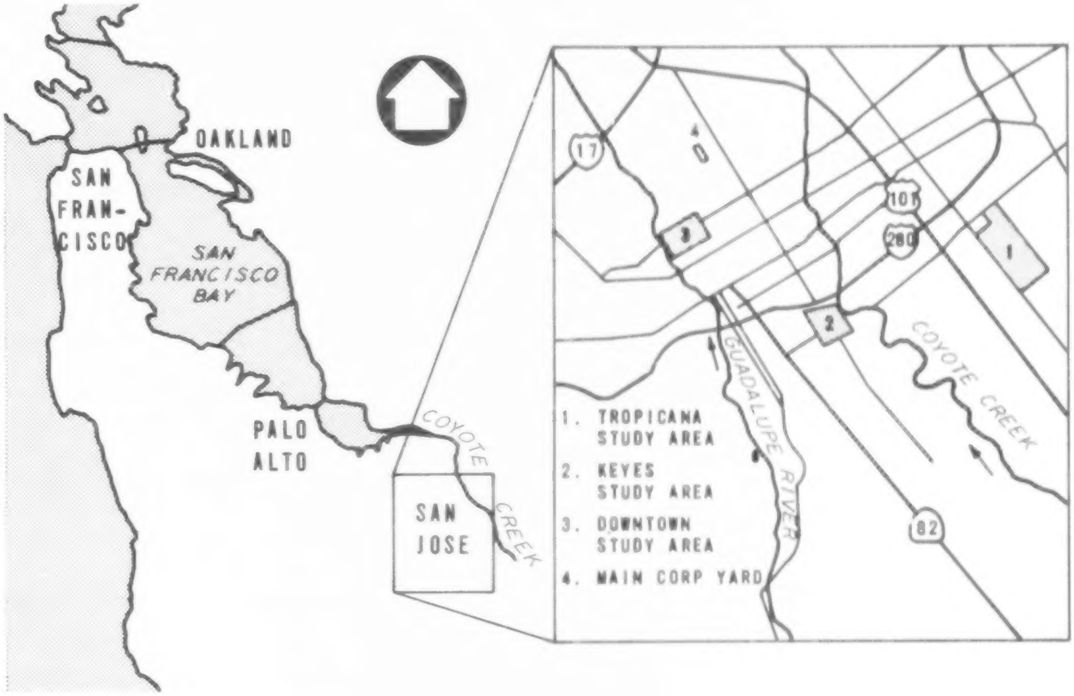


Figure 52. San Jose and the three study areas.

The Tropicana area covers about 79 ha (195 acres) and is mostly residential. The area includes a portion of a large shopping center and is adjacent to three schools. There are few vacant lots, some roadside trees, and no construction activities. Some streets have heavy traffic, but most carry light traffic. Stormwater is eventually discharged into Silver Creek, a tributary of Coyote Creek. The area is normally swept every 5 weeks.

The Keyes Street area covers about 37 ha (92 acres) and its major land use is residential, with some strip commercial use. The study area is adjacent to several schools and playing fields. This area has few vacant lots, many roadside trees, and no construction. Several streets have heavy traffic, but most have light traffic. The stormwater is discharged directly into Coyote Creek. This area is also normally swept every 5 weeks.

The downtown area covers about 40 ha (100 acres). Its major land uses are commercial and industrial, with some older single- and multiple-family residential areas, much roadside vegetation, and many vacant lots (previously cleared for redevelopment). There is some construction and several streets have heavy traffic. The stormwater from this area is discharged directly into the Guadalupe River. The downtown commercial part of the study area is normally swept daily, and the remainder of the area is normally swept every 5 weeks.

More detailed information about the study areas is provided in Tables 67, 68, and 69. In all the areas, the curb types are 90° straight-edged concrete, the topography is flat, and the ambient air conditions and soils are similar.

Table 67. GENERAL CHARACTERISTICS OF THE THREE STUDY AREAS

| | Study area | | |
|---|---|---|---|
| Characteristics | Tropicana | Keyes | Downtown |
| Drainage area, acres | 195 | 92 | 100 |
| Curb length, miles | 12.7 | 5.4 | 7.0 |
| Number of inlets | 55 | 17 | 25 |
| Inlets/curb-mile | 4.3 | 3.2 | 3.6 |
| Inlets/acre | 0.28 | 0.18 | 0.25 |
| Acres/curb-mile | 15 | 17 | 14 |
| Land use | Residential, low income, built 1960, some commercial, adjacent to 3 schools | Commercial and older residential, adjacent to school and playing fields | Commercial, industrial, older residential |
| Vacant lots in area | Few | Few | Many |
| Construction in area | None | None | Some |
| Traffic density | Light to heavy | Light to heavy | Light to heavy |
| Notes: | Good air sampling sites; moderate vegetation, few large trees; and minimal leaf removal problem | Adjacent to college stadium and subject to heavy traffic and parking periodically | Air quality sampling point |

## Table 68. SURFACE AREA AND LAND USE IN THE STUDY AREAS

| | Percent of study areas | | |
|---|---|---|---|
| | Downtown | Keyes | Tropicana |
| Surface area | | | |
| Rooftops (<3 stories tall) | 24.0 | 17.0 | 16.5 |
| Rooftops (>3 stories tall) | 2.0 | 0 | 0 |
| Lawn/landscaped area | 1.0 | 39.0 | 37.0 |
| Vacant space | 48.0 | 21.0 | 28.5 |
| Sidewalks | 4.0 | 4.0 | 4.0 |
| Street | 21.0 | 19.0 | 14.0 |
| | 100.0 | 100.0 | 100.0 |
| Land use | | | |
| Commercial | 33.0 | 11.0 | 0 |
| Residential | 2.0 | 86.0 | 83.0 |
| Industrial | 31.0 | 0 | (some) |
| Other (institutional, vacant land, etc.) | 34.0 | 3.0 | 17.0 |
| | 100.0 | 100.0 | 100.0 |

## Table 69. ESTIMATED DAILY TRAFFIC VOLUMES IN THE TEST SITES

| Test sites | Weighted average daily traffic[a] | Estimated minimum daily traffic[b] | Estimated maximum daily traffic[c] |
|---|---|---|---|
| Tropicana - good asphalt streets | 2,200 | 100 | 18,000 |
| Keyes - overall | 4,600 | 50 | 26,000 |
| Keyes - good asphalt streets | 8,300 | 200 | 26,000 |
| Keyes - oil and screens streets | 200 | 50 | 1,000 |
| Downtown - overall | 7,700 | 500 | 25,000 |
| Downtown - good asphalt streets | 10,000 | 1,500 | 25,000 |
| Downtown - poor asphalt streets | 2,800 | 500 | 7,500 |

a. Estimates based on field measurements. Weighted by representative street segment lengths.

b. Minimum estimated daily traffic for any one street segment in test area.

c. Maximum estimated daily traffic for any one street segment in test area.

Preliminary street surface particulate sampling suggested that sites with significantly different street surfaces should be treated separately in the study. For this reason, the Keyes Street and downtown study areas were each divided into two test sites. The Tropicana study area was best treated as a single test site. Thus, a total of five test sites were used in the initial field activities [1]:

- Tropicana - good asphalt street test site
- Keyes Street - good asphalt street test site
- Keyes Street - oil and screens street test site
- Downtown - good asphalt street test site
- Downtown - poor asphalt street test site

Typical street scenes within the five test sites are shown in Figure 53.

Buffer zones were established around each study area. The buffer zones were swept at the same time and with the same number of passes as the test sites to prevent excessive tracking or blowing of street pollutants into the study areas. The buffer zones around the Keyes Street study area are identified in Figure 54; the locations of the two different types of street surfaces (test sites) in this area are also indicated.

Although samples of street surface pollutants were initially collected from all three study areas, an illegal discharge in the downtown area required that the area be eliminated from further study. Most of the later studies were conducted in the Keyes Street and Tropicana study areas.

Climatic features of most interest to street cleaning operations are rainfall, low temperatures, and snow. San Jose's mean annual precipitation is 33 cm (13 in.), most of which occurs from November through April. The normal monthly temperature ranges from 9.6 to 20°C (49 to 68°F); on the average, there are very few days in which temperatures remain below freezing, and snow is extremely rare in San Jose.

### Problem Assessment

Pollutant characteristics, sources, and accumulation rates were studied in depth to help identify the problem of urban runoff control from street surfaces. Special, accurate, long-term pollutant monitoring techniques were developed for the project [1]. These techniques can easily be used by a public works department to monitor a large area in a relatively short time with readily available equipment. The sampling equipment in the San Jose study included a signalized truck with a CB radio and a trailer carrying a 5 kW generator and heavy duty wet-dry vacuum units with accessories and tools.

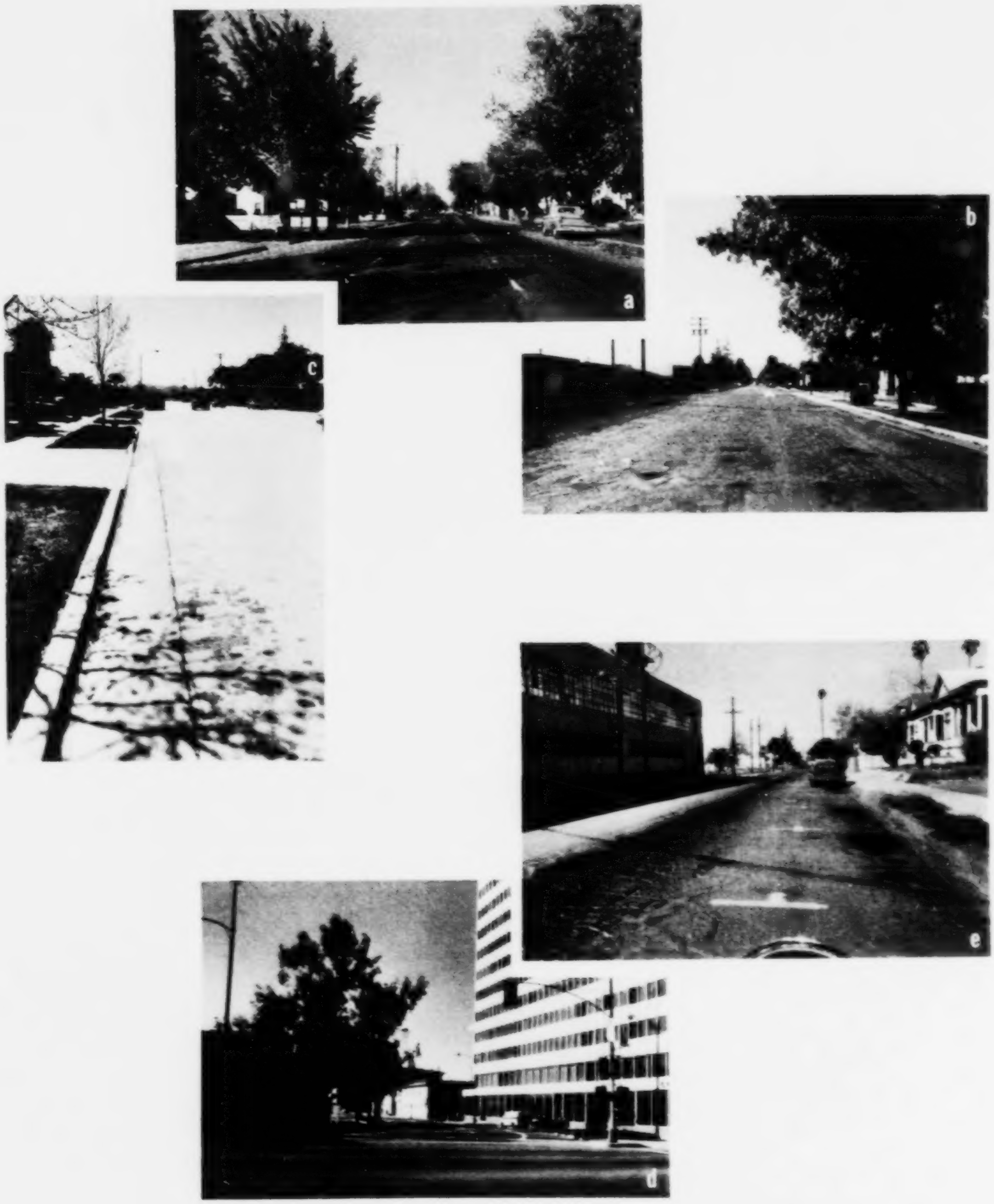


Figure 53. Typical streets in the five San Jose test sites: (a) Keyes, good asphalt surface, (b) Keyes, oil and screens surface, (c) Tropicana, good asphalt surface, (d) downtown, good asphalt surface, (e) downtown, poor asphalt surface [1]

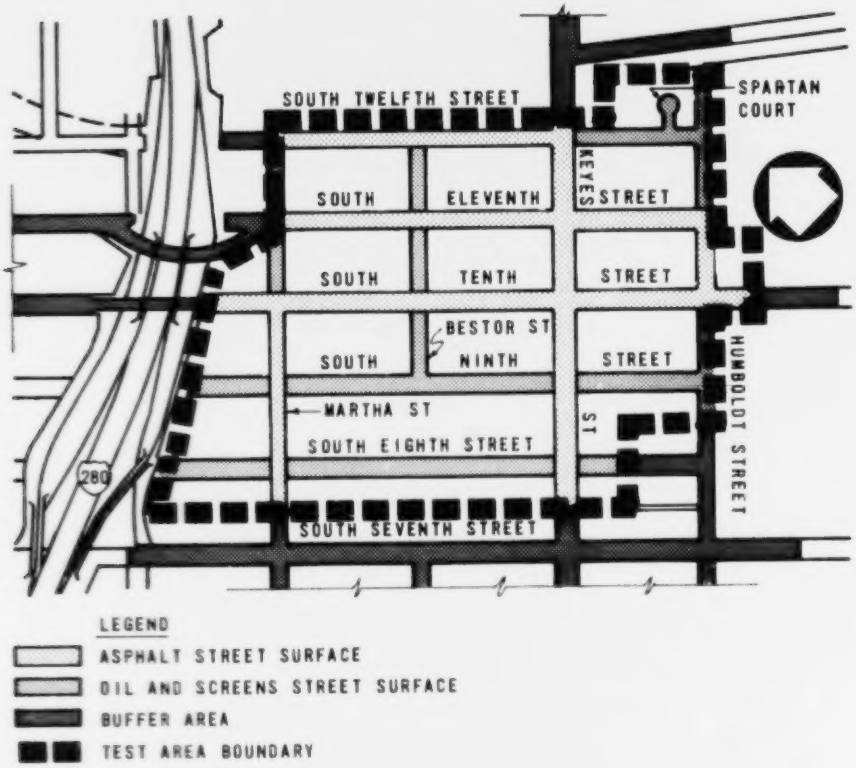


Figure 54. Keyes Street buffer zone and test sites [1].

Pollutant Variations with Particle Size--

Because the chemical composition of different particle sizes on street surfaces can vary significantly, all San Jose samples were divided into the following eight particle size ranges before chemical analysis:

| Particle size, microns (μ) |
|---|
| <45 |
| 45-106 |
| 106-250 |
| 250-600 |
| 600-850 |
| 850-2,000 |
| 2,000-6,370 |
| >6,370 |

The chemical and particle size information was used to determine the accumulation rates and street cleaning equipment performance for the different pollutants.

Almost all of the pollutants in the test areas showed higher concentrations with decreasing particle size. Mercury, cadmium, zinc, lead, Kjeldahl nitrogen, and orthophosphates showed the highest concentrations with the smaller particle sizes. However, copper and chromium had the lowest concentrations with the smaller particle sizes.

These data indicate that conventional street cleaning methods, which are most effective in removing large particle sizes, may be unable to remove enough of those pollutants found mainly in the smaller quantities of smaller particle sizes to completely meet pollutant control objectives, unless extra effort is expended. Street cleaning can remove useful amounts of these pollutants, however, because they are also found in the bigger quantities of larger particle sizes.

Pollutant Concentrations in Runoff--

Stormwater runoff from three San Jose storms was sampled and analyzed to determine pollution concentrations. The results for major pollutants are summarized in Tables 70 and 71.

Table 70. POLLUTANT CONCENTRATIONS IN STORM RUNOFF

| | | | Average pollutant concentration, mg/L | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | Storm | | Solids | | Organics | | Nutrients | | Heavy metals | | |
| Study area | Date | No. of samples | SS | VSS | $BOD_5$ | COD | TKN | $OPO_4$ | Pb | Zn | Cr |
| Keyes | 3/15-16/77 | 16 | 112 | -- | 30 | 133 | 8.0 | 3.3 | 0.27 | 0.11 | 0.01 |
| | 3/23-24/77 | 8 | 571 | 140 | 22 | 350 | 3.6 | -- | 0.76 | 0.32 | 0.03 |
| | 4/30-5/1/77 | 1[a] | 75 | -- | -- | -- | -- | 1.7 | -- | -- | -- |
| Tropicana | 3/15-16/77 | 21 | 164 | -- | 25 | 77 | 3.1 | 2.2 | 0.22 | 0.10 | 0.01 |
| | 3/23-24/77 | 25 | 120 | 27 | 17 | 160 | 3.8 | 0.5 | 0.20 | 0.12 | 0.01 |
| | 4/30-5/1/77 | 10 | 220 | -- | 28 | 260 | 15.0 | 6.0 | 0.66 | 0.27 | 0.02 |
| | Average[b] | | 160 | 32 | 25 | 114 | 5.2 | 2.6 | 0.27 | 0.12 | 0.01 |

a. Partial storm.

b. Average concentrations are flow weighted.

BOD was a key water quality parameter, and 20-day incubation tests unexpectedly suggested that settled materials could exert a long-term oxygen demand much larger than the initial effects of BOD discharges.

The recommended water quality criteria for various beneficial uses were exceeded by the concentrations of a number of pollutants in the urban runoff; these are summarized in Table 72. Of all the beneficial uses, aquatic life may be harmed by the greatest number of pollutants. Street cleaning can remove portions of these troublesome pollutants from source areas before rains wash them into the receiving waters.

### Table 71. POLLUTANT STRENGTHS IN STORM RUNOFF

| | | | Average pollutant strength, mg/g total solids | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | Storm | | Solids | | Organics | | Nutrients | | Heavy metals | | |
| Study area | Date | No. of samples | SS | VSS | $BOD_5$ | COD | TKN | $OPO_4$ | Pb | Zn | Cr |
| Keyes | 3/15-16/77 | 16 | 770 | -- | 204 | 911 | 55 | 23 | 1.80 | 0.75 | 0.068 |
| | 3/23-24/77 | 8 | 840 | 210 | 32 | 520 | 5 | -- | 1.10 | 0.47 | 0.044 |
| | 4/30-5/1/77 | 1[a] | 480 | -- | -- | -- | -- | 11 | -- | -- | -- |
| Tropicana | 3/15-16/77 | 21 | 596 | -- | 92 | 280 | 11 | 8 | 0.80 | 0.36 | 0.040 |
| | 3/23-24/77 | 25 | 430 | 96 | 61 | 570 | 14 | 2 | 0.71 | 0.42 | 0.033 |
| | 4/30-5/1/77 | 10 | 580 | -- | 74 | 680 | 39 | 16 | 1.70 | 0.71 | 0.050 |
| | Average[b] | | 593 | 97 | 94 | 421 | 19 | 10 | 1.01 | 0.45 | 0.043 |

a. Partial storm.

b. Averages are weighted by total solids loads.

### Table 72. RUNOFF WATER QUALITY PARAMETERS EXCEEDING RECOMMENDED BENEFICIAL USE CRITERIA [1]

| Beneficial use | Parameters exceeding recommended criterion |
|---|---|
| Aquatic life | Cr, Cd[a], Pb[a], Hg[a], $BOD_5$, turbidity[a], SS[a] |
| Marine life | $PO_4$[a], Cd, Cu, Zn |
| Freshwater for public supply | Cd, Pb[a] |
| Livestock | Pb[a] |
| Recreation | $PO_4$[a] |
| Irrigation | Cd |
| Wildlife | None |

a. Exceeded criteria by more than 10 times.

## Pollutant Sources--

On a weight basis, the quantity of the contaminants found on street surfaces depends on local geologic conditions, with added fractions resulting from motor vehicle wear and emissions and from inputs from surrounding areas. The wear of street surfaces normally makes only minor contributions. The more common street surface pollutants and their sources are listed in Table 73.

Table 73. SOURCES OF COMMON STREET SURFACE POLLUTANTS

| Source | Pollutant |
|---|---|
| Local soil erosion | Particulates (inert) |
| Local plants and soils (transported by wind and traffic) | Nitrogen and phosphorus |
| Wear of asphalt street surfaces | Phenolic compounds |
| Spills and leaks from vehicles | Grease, petroleum, n-paraffin, lead |
| Spills from vehicles (oil additives) | Phosphorus and zinc |
| Combustion of leaded fuels | Lead |
| Tire wear | Lead, zinc, and asbestos |
| Wear of clutch and brake linings | Asbestos, lead, chromium, copper, and nickel |
| Wear of vehicle moving metal parts | Copper, nickel, and chromium |
| Deicing compounds (traffic dependent); possibly also roadway abrasion and local soils | Chlorides |

A comparison of pollutant runoff yield and street surface loading differences during storms revealed notable differences. COD, Kjeldahl nitrogen, and orthophosphates occurred in much greater proportions (about 3 to 180 times) in the total runoff than from the street dirt, indicating that erosion from offstreet areas (such as parking lots, landscaped areas, vacant land, rooftops) during storms is probably responsible for most of the organic and nutrient yields. Thus, if organics and nutrients must be significantly reduced in the runoff, street cleaning by itself may not be sufficient.

### Pollutant Accumulation Rates--

The accumulation rates of street surface contaminants must be known to understand the magnitude of the problem a street cleaning program must address and to determine the most effective control methods.

The San Jose study showed that the accumulation rates varied widely in the different test sites. The deposition and accumulation dynamics of street contaminants seem to be governed primarily by geographical location, season, period of time since last swept or the last rain, land use, street surface type and condition, and conditions in the area, such as the presence of vacant lots, commercial development, pedestrian and automobile traffic, parking, and ambient and traffic winds. The specific dynamics are, therefore, a function of many site conditions that can vary widely. Such variations should be considered in scheduling street cleaning programs for different types of areas.

Earlier nationwide studies have shown that total solids accumulation rates vary over a very wide range, 0.8 to 762 kg/curb-km·d (3 to 2,700 lb/curb-mi·d) [4]. Local monitoring programs, therefore, are essential to understand the complex influencing factors before designing a comprehensive street cleaning program.

In San Jose, the lowest overall pollutant accumulation rates occurred in the oil and screens test site, which also always retained the highest surface loadings. The downtown test site had the highest accumulation rates of the five sites, thought to be due to the poor condition of the asphalt streets and the characteristics of the area.

Over time, the accumulation rate (deposition rate minus removal rate) varied in a sawtooth pattern of deposition and removal as shown in Figure 55. The deposition rate depends on the area characteristics, while removal can occur by street cleaning or by winds or rain. Without removal, the particulate loading levels off over time, after 1 to 3 months. This effect is thought to be caused by wind and automobile-related air turbulence suspending the particles in the air. These patterns should be identified and considered in establishing optimum street cleaning frequencies. However, longer periods between street cleaning may not significantly increase loadings, although they could increase roadside airborne particulate concentrations.

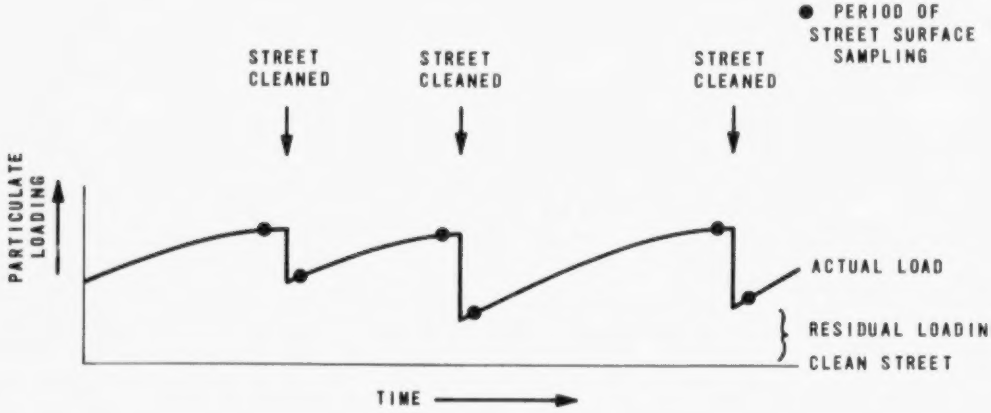


Figure 55. Sawtooth pattern of particulate deposition and removal [1].

The pollutant removal capabilities of storms were studied because of their effect on the loadings remaining on the streets after rain. The monitored storms had a much smaller removal effect in the oil and screens test site than in the test sites with asphalt streets, presumably because of the trapping action of the rough oil and screens surface. In general, smaller storms left the street surfaces cleaner than larger ones, which probably washed large amounts of eroded materials onto the streets. The smooth asphalt surfaces were almost completely cleaned by the less intense storms.

### Countermeasure Philosophy

The purpose of street cleaning as a pollution abatement measure is to limit the buildup of pollutants at the source, i.e., on street surfaces, and thereby control their migration into receiving waters and the atmosphere. A street cleaning program that is most cost effective in meeting its environmental objectives should consider the interrelationships between area characteristics and management options. The management options with this control measure are the selection of cleaning equipment and a cleaning program.

#### Street Cleaning Equipment--

Different street cleaning equipment and their use are described in Table 74. The purpose of the demonstration project, however, was not to compare specific types of street cleaning equipment, but to determine the range and capabilities of street cleaning equipment in general and under various "real-world" operating conditions. Several street cleaning programs were evaluated under various operating conditions and cleaning frequencies, using the following three types of street cleaning equipment:

- Four-wheel mechanical street cleaner
- State-of-the-art mechanical street cleaner
- Vacuum-assisted street cleaner

#### Street Cleaning Program--

Before designing a street cleaning program as a BMP, there should be good evidence that street cleaning is either the most appropriate approach or can be combined in an integrated control measure approach to combat the specific pollutant problems. The potential multiuse benefits or tradeoffs must be considered, i.e., clean streets and water pollution control. This requires first determining the sources of the problem pollutants.

If the objective of a street cleaning program is to remove the most pollutants from the runoff, then an appropriate program could simply involve cleaning the streets with the highest unit removal rates (load removed per pass) and keeping the number of passes a year for other streets to a minimum. No service objectives are this simple, however, and more complex program design techniques are usually necessary.

The following three steps briefly summarize the procedure recommended by the San Jose demonstration project to design a street cleaning program.

1. Determine an allowable street surface residual loading (load/curb length) from the city's street cleaning objectives. These objectives are determined by environmental, aesthetic, safety, and public relations requirements to meet urban runoff load allocations and fugitive dust emission allocations, to control debris and oil accumulation in traffic lanes, and to reduce service area complaints.

2. Measure or estimate the long-term average particulate accumulation rate on street surfaces. This will vary with type of street surface and with street cleaning frequencies.

3. Determine the maximum allowable effective days of accumulation (EDA) from charts, and then determine the needed combinations of cleaning interval and cleaning efficiency. The use of the charts and the impact of frequent rains on the procedure are described in the demonstration project final report [1].

Because of the interdependence of some variables, this procedure should be checked and repeated periodically.

Table 74. TYPES OF AVAILABLE STREET CLEANING EQUIPMENT

| Type | Characteristics | Use |
|---|---|---|
| Mechanical street cleaner | Rotating brooms, plus water spray to control dust. Dirt is transported to storage hopper on moving conveyor. May be self-dumping, 3- or 4-wheel. | Used for most street cleaning in most U.S. communities. |
| Vacuum-assisted mechanical street cleaner | Vacuum system transports dirt from rotating brooms to hopper. Transported dirt is saturated with water. | Used in Europe for many years. Has seen limited use in U.S. for some time. |
| Regenerative street cleaner | Recycled air blasts dirt and debris from road surface into hopper; air is then regenerated through dust separation system. | Relatively minor use. |
| Small, industrial-type vacuum sweeper | Vacuum is applied directly to street. | Most useful for parking lots, sidewalks, factory floors. Of limited use on city streets. |
| Hand sweeping | Push cart or motor scooter and hand tools. | To back up machines, and for areas machines cannot reach, particularly around parked cars in business districts. |
| Street flusher | Water tank, pressure supply, and three or more individually controlled nozzles. | Mostly for aesthetic purposes. Generally (and preferably) used to quickly displace dirt and debris from traffic lanes to gutter. Up to 22 ft wide street on one pass. Has potential problems with transport rates and volumes, and if pollutants enter storm sewer they might be flushed into the receiving water. |

## OPERATION AND PERFORMANCE

The purpose of the San Jose demonstration project was to determine the range and capabilities of street cleaning equipment in general. Very little data showed any significant differences in the performance of different types of street cleaning equipment.

### Project Operation

#### Equipment Operation--

Specific equipment maintenance was not monitored in detail in the San Jose project, but the maintenance costs were and were included in the street cleaning cost analysis. A survey of cities nationwide found the following equipment components to be the most subject to wear, in descending order: brushes, conveyor and elevator drives, tires, elevators, flights, hydraulic systems, and transmissions [5].

Although synthetic broom materials last longer (by curb length swept) than steel or natural fibers, broom life is not the most important factor. The goal is removal effectiveness, and this has been shown to depend on broom fiber, brush speed, pattern (strike), and forward speed. Half the cleaning equipment surveyed was operated with a main broom rotational speed between 1,500 and 2,000 r/min and with a strike of 10 to 15 cm (4 to 6 in.) [6]; all these determinations must depend on many local conditions.

#### Cleaning Frequency--

The cleaning frequencies used in the San Jose study ranged from two passes every day to fewer than one pass every month. Each piece of equipment was evaluated in the field in two different 7 week periods or phases: once during the winter and once during the summer (with the exception of the vacuum-assisted cleaner).

During the first 2 weeks of each phase, cleaning every weekday was tested with single passes the first week and two passes the next week. During the last 5 weeks of each phase, weekly cleaning intervals were tested. The equipment was rotated through the different test sites at the end of each phase. This schedule allowed the different characteristics and long-term seasonal differences in the test sites to be evaluated in addition to the range of equipment effectiveness.

Adjacent buffer zones, up to three times the size of the test sites, were also cleaned to reduce potential edge effects (tracking of particulates into the test sites from the adjacent areas).

The long-term and frequent sampling in the test sites measured contaminant accumulation rates on street surfaces and identified the range of performance that may be expected from currently available street cleaning equipment.

## Performance

The street surface condition, initial loading (total loading value and particle size distribution), and various other environmental factors affect street cleaning performance. Equipment operation variables that most affect street cleaning performance include the number of passes and the street cleaning interval.

The most important measure of cleaning effectiveness is the load per curb length removed. This removal value and the unit curb length costs are used to calculate the cost for removing a unit load of pollutant for a specific street cleaning program. The often-used effectiveness measure, percentage of the initial loading removed, is very misleading because it does not measure the magnitude of the amount of material removed.

It must be stressed that the performance of street cleaning equipment as measured in the San Jose tests may not adequately indicate how well the equipment operates under other conditions.

### Equipment Effectiveness--

Street cleaning equipment performance results, obtained by comparing San Jose street surface samples collected just before and just after cleaning, are presented in Table 75. These results cover 26 different test conditions, representing different test sites, equipment types, seasons, number of passes, and cleaning intervals. The two measures of cleaning effectiveness, load per curb length (preferred) and percentage of initial loading removed (misleading), are presented.

From statistical tests on the performance data, the characteristics of the area to be cleaned (street surface conditions and accumulation rates) and, in most cases, the street cleaning program (number of passes and cleaning intervals), were found to be more important than the selection of the type of street cleaning equipment. Other considerations, such as maneuverability, life-cycle costs, hopper capacity, etc., may be important when selecting equipment.

### Program Effectiveness--

The design of an effective street cleaning program should incorporate local conditions and operating procedures that may affect the pollutant removal efficiency. The number of passes and the cleaning interval influence removal effectiveness, and other considerations, such as particle size distribution, expected pollutant removal effectiveness, loading distribution, and potential for redistribution, should be evaluated.

Number of Passes--In most cases, two passes with the same piece of equipment removed a larger quantity of material from the street than did a single pass, as expected. Exceptions were found, however, in some of the tests on oil and screens streets. Here, two passes per day with the state-of-the-art mechanical four-wheel street cleaner resulted in a higher residual loading on the street surface than before the tests. This may be due to the extra

## Table 75. STREET CLEANING PERFORMANCE DURING SAN JOSE DEMONSTRATION PROJECT

| | | | | | Before street cleaning | | | After street cleaning | | | Cleaning effectiveness | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Test area | Number of passes | Approximate cleaning interval | Equipment type[a] | Test phase[b] | Median particle size, μ | Bulk density, $g/cm^3$ | Total solids loading, lb/curb-mi | Median particle size, μ | Bulk density, $g/cm^3$ | Total solids loading, lb/curb-mi | Total solids removed, lb/curb-mi | Percentage of initial loading removed | Hopper contents median particle size, μ[c] |
| Tropicana - good asphalt streets | 2 | Daily | C | W | 410 | 0.82 | 328 | 320 | 1.06 | 132 | 196 | 60 | 3,190 |
| | 1 | Daily | A | W | 965 | 1.30 | 115 | 430 | 1.20 | 98 | 17 | 13 | 3,170 |
| | 1 | Daily | B | W | 430 | 1.12 | 350 | 300 | 1.10 | 165 | 185 | 53 | 2,090 |
| | 1 | Daily | B | S | 295 | 0.95 | 200 | 275 | 1.05 | 116 | 84 | 42 | 2,760 |
| | 1 | Daily | C | S | 380 | 0.98 | 206 | 420 | 1.15 | 113 | 92 | 45 | 3,120 |
| | 1 | Weekly | A | W | 510 | 0.98 | 164 | 450 | 1.15 | 87 | 77 | 47 | 5,750 |
| | 1 | Weekly | B | S | 325 | 0.98 | 207 | 300 | 1.02 | 128 | 78 | 38 | 3,460 |
| | 1 | Weekly | C | S | 420 | 0.86 | 221 | 435 | 1.02 | 117 | 104 | 47 | 2,280 |
| Keyes - good asphalt streets | 2 | Daily | A | W | 480 | 0.98 | 401 | 460 | 1.10 | 258 | 144 | 36 | 940 |
| | 2 | Daily | B | W | 450 | 1.00 | 173 | 340 | 1.07 | 142 | 32 | 19 | 5,520 |
| | 2 | Daily | B | S | 420 | 1.03 | 317 | 450 | 1.17 | 201 | 116 | 37 | 790 |
| | 2 | Daily | C | S | 470 | 0.85 | 436 | 500 | 0.90 | 374 | 625 | 14 | 2,260 |
| | 1 | Weekly | B | W | 520 | 0.87 | 381 | 390 | 0.97 | 294 | 87 | 23 | 4,550 |
| | 1 | Weekly | B | S | 555 | 0.98 | 512 | 510 | 0.98 | 350 | 162 | 32 | 3,280 |
| | 1 | Weekly | C | W | 510 | 0.78 | 459 | 390 | 0.90 | 295 | 165 | 36 | 4,460 |
| | 1 | Weekly | C | S | 560 | 0.94 | 548 | 490 | 1.18 | 291 | 25 | 47 | 4,720 |
| Keyes - oil and screens streets | 2 | Daily | A | W | 560 | 1.35 | 2,654 | 600 | 1.38 | 2,208 | 445 | 17 | 940 |
| | 2 | Daily | B | W | 650 | 1.30 | 1,830 | 570 | 1.28 | 1,930 | -98 | -6 | 5,940 |
| | 2 | Daily | B | S | 480 | 1.15 | 1,244 | 460 | 1.20 | 1,141 | 104 | 8 | 835 |
| | 2 | Daily | C | S | 540 | 1.18 | 2,056 | 520 | 1.20 | 2,078 | -22 | -1 | 2,260 |
| | 1 | Weekly | B | W | 670 | 1.37 | 2,370 | 600 | 1.30 | 1,860 | 510 | 22 | 4,550 |
| | 1 | Weekly | B | S | 490 | 1.08 | 1,489 | 485 | 1.16 | 1,318 | 164 | 11 | 3,280 |
| | 1 | Weekly | C | W | 930 | 1.15 | 2,200 | 660 | 1.20 | 2,030 | 171 | 8 | 4,460 |
| | 1 | Weekly | C | S | 550 | 1.06 | 1,840 | 530 | 1.26 | 1,730 | 110 | 6 | 4,720 |
| Downtown - good asphalt streets | 1 | Daily | C | W | 430 | 0.99 | 243 | 380 | 1.03 | 160 | 83 | 34 | 2,660 |
| Downtown - poor asphalt streets | 1 | Daily | C | W | 570 | 0.90 | 1,350 | 530 | 0.98 | 808 | 543 | 40 | 2,660 |

a. Equipment types: A = 4 wheel vacuum-assisted mechanical street cleaner; B = state-of-the-art 4-wheel mechanical street cleaner; C = 4-wheel mechanical street cleaner.

b. Test phases; W = winter, S = summer.

c. Hopper samples from the Keyes and Downtown test areas were not separated by type of street surface before particle size analysis.

erosion caused by excessive mechanical action of the broom on the weaker oil and screens surface.

Cleaning Interval--The effect of cleaning interval on total annual solids removal is summarized in Figure 56. The curves, idealized for different street surfaces in this plot, show decreased removal quantities per equipment pass as the number of passes per year increases. By increasing from 10 to 100 passes per year, the potential effort and cost can increase by 10 times, whereas the amount removed only increases by a factor of about 3 to 5.

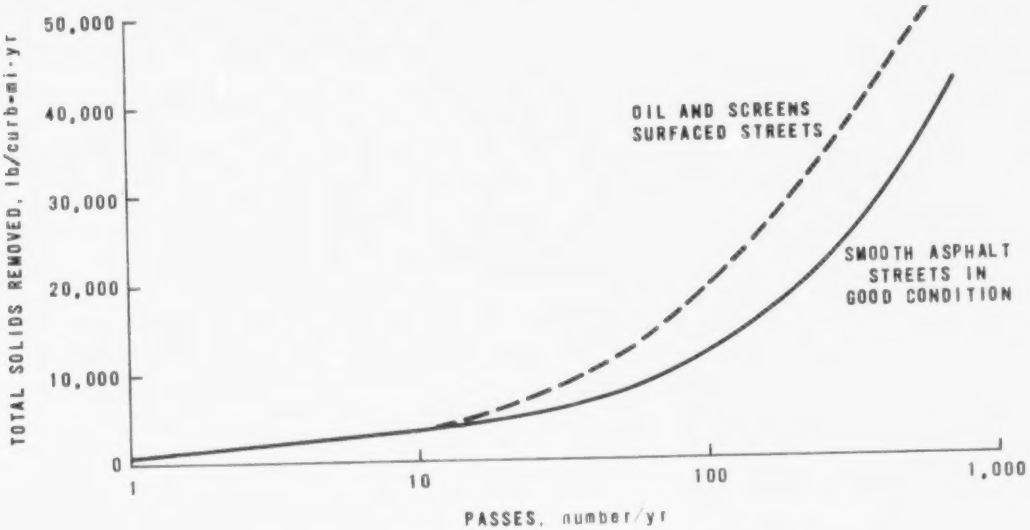


Figure 56. Variation of annual solids removal with number of equipment passes [1].

Particle Size Removals--The variation of removal efficiency with particle size is important because pollutant concentrations can vary with particle size. Particle size removals for the five San Jose street conditions, for all street cleaning programs combined, are shown in Figure 57.

The effectiveness of street cleaners generally increases with particle size, although there are quite large variations. This efficiency increase is confirmed by the decrease in the street surface median particle size after cleaning, and by hopper median particle sizes always being much larger than the median particle sizes on the street surface before cleaning (Table 75). Thus, there is a larger fraction of smaller particles on the street after street cleaning than before.

Pollutant Removal--The pollutant removals from San Jose streets are given in Table 76 for each of the five test sites. The figures represent averages for all street cleaning programs combined. The percentage removals for the various pollutants were about the same as the total solids percentage removals. They ranged from 20 to 50% for all pollutants and test sites, except at the oil and screens test site. The mass removal rates at a given

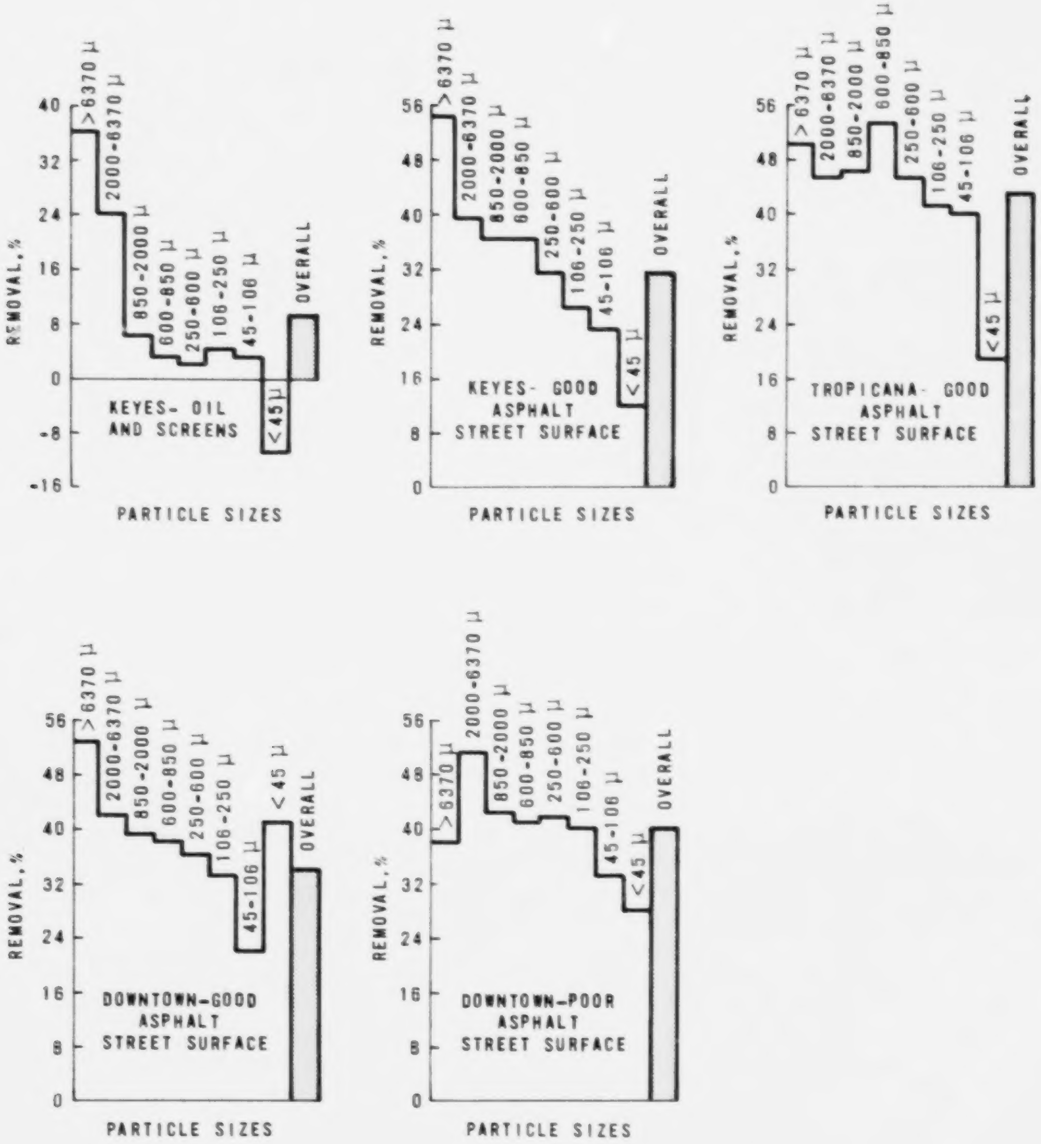


Figure 57. Total solids removal by particle size, from various street surfaces [1].

site varied greatly, however, because of the large variation in initial loadings for these pollutants.

Table 76. REMOVAL EFFECTIVENESS FOR VARIOUS POLLUTANTS BY TEST SITE (AVERAGES FOR ALL STREET CLEANING PROGRAMS COMBINED) [1]

| | Keyes - oil and screens test site | | Keyes - good asphalt street test site | | Tropicana - good asphalt streets test site | | Downtown - poor asphalt street test site | | Downtown - good asphalt streets test site | |
|---|---|---|---|---|---|---|---|---|---|---|
| Pollutant | Amount removed, lb/curb mi | Removal, % | Amount removed, lb/curb mi | Removal, % | Amount removed, lb/curb mi | Removal, % | Amount removed, lb/curb mi | Removal, % | Amount removed, lb/curb mi | Removal, % |
| Total solids | 170 | 9 | 130 | 33 | 100 | 43 | 540 | 40 | 83 | 34 |
| Chemical oxygen demand | 12 | 9 | 16 | 33 | 9.7 | 46 | 61 | 40 | 11 | 32 |
| Kjeldahl nitrogen | 0.14 | 6 | 0.28 | 32 | 0.21 | 47 | 1.3 | 38 | 0.16 | 33 |
| Ortho-phosphate | 0.0089 | 7 | 0.018 | 31 | 0.017 | 44 | 0.079 | 38 | 0.012 | 32 |
| Lead | 0.15 | 5 | 0.81 | 30 | 0.40 | 44 | 1.0 | 37 | 0.49 | 31 |
| Zinc | 0.066 | 12 | 0.079 | 29 | 0.049 | 45 | 0.27 | 39 | 0.072 | 31 |
| Chromium | 0.071 | 9 | 0.051 | 32 | 0.039 | 50 | 0.24 | 42 | 0.047 | 36 |
| Copper | 0.13 | 13 | 0.081 | 34 | 0.072 | 51 | 0.50 | 43 | 0.039 | 38 |
| Cadmium | 0.00024 | 8 | 0.00030 | 30 | 0.00027 | 45 | 0.0015 | 40 | 0.0023 | 49 |

Loading Distributions--Cumulative loading distributions across three different street surfaces are compared in Figure 58. Only about 60% of the total solids fell within the normal 2.4 m (8 ft) path of a street cleaner on the oil and screens surface, while over 95% of the total solids loading could be reached on the asphalt surfaces. These distributions can be further modified by car parking conditions.

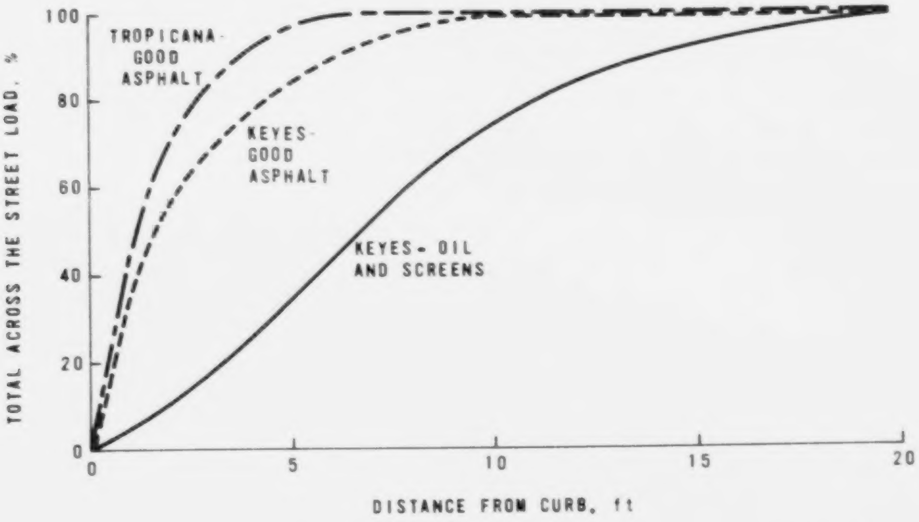


Figure 58. Cumulative loading distributions across streets with different surfaces [1].

A study of solids distribution across the street by particles size found about an even distribution in the Tropicana-good asphalt test site, no clear trends in the Keyes-good asphalt test site, and more large than fine particles near the curb in the Keyes oil and screens test site.

Loading Redistribution--Street cleaning redistributes surface materials across the street. Average distributions before and after cleaning at three of the five San Jose test sites are compared in Figure 59. On the smoother surfaces (Tropicana-good asphalt and Keyes-good -asphalt), the peak load at the curb was reduced by about 80%. Some areas show increases in solids loadings caused by broom action and turbulence partially redistributing the high curbside loading out into the street. The rougher oil and screens streets had much higher loadings in the center of the street; as a result, removals on a load per unit area basis were much more uniform across this surface.

Effects of Parking--Vehicles parked along a street cleaning route reduce the length of curb that can be cleaned. In addition to the curb length blocked by a car, front and back clearances also cannot be cleaned because of the turning radius of the street cleaner. Since most of the street surface pollutants are close to the curb on smoother streets with little parking, parked vehicles can drastically reduce the cleaning effectiveness of normal cleaning programs on these streets. Parking restrictions may be used to improve cleaning effectiveness.

Parking can also strongly affect the accumulative loading distribution across a street surface. The effects of different parking conditions on solids distributions across two different street surfaces are shown in Figure 60. Extensive parking blocks the migration of particulates toward the curb, resulting in higher middle-of-the-street loadings than on streets with little or no parking. Further, this blocking effect is much more noticeable on smooth asphalt surfaces than on oil and screens surfaces, whose roughness itself seems to control particle migration.

Parking Restrictions--Parking restrictions, which allow street cleaners access to the curb, usually (but not always) improve cleaning effectiveness. The effects of parking restrictions on removal effectiveness, for various street surfaces and parking densities, are shown in Figure 61.

In many situations, parking restrictions will about double the removal effectiveness; in most situations, they will improve it. However, on good asphalt streets with extensive 24 hour a day onstreet parking (as in high density residential neighborhoods), most of the solids load will not fall within the 2.4 m (8 ft) strip next to the curb. If more than about 80% of the curb length is occupied by parked cars, it would be more effective to clean around the parked cars (i.e., without parking restrictions), as shown in Figure 61. Similarly, it should be more effective to clean around the parked cars if more than about 95% of the curb length is occupied on good asphalt streets with extensive daytime only parking, or likewise on oil and screens surfaced streets. Of course, periodic curbside cleaning would still be needed to remove sidewalk-originating litter.

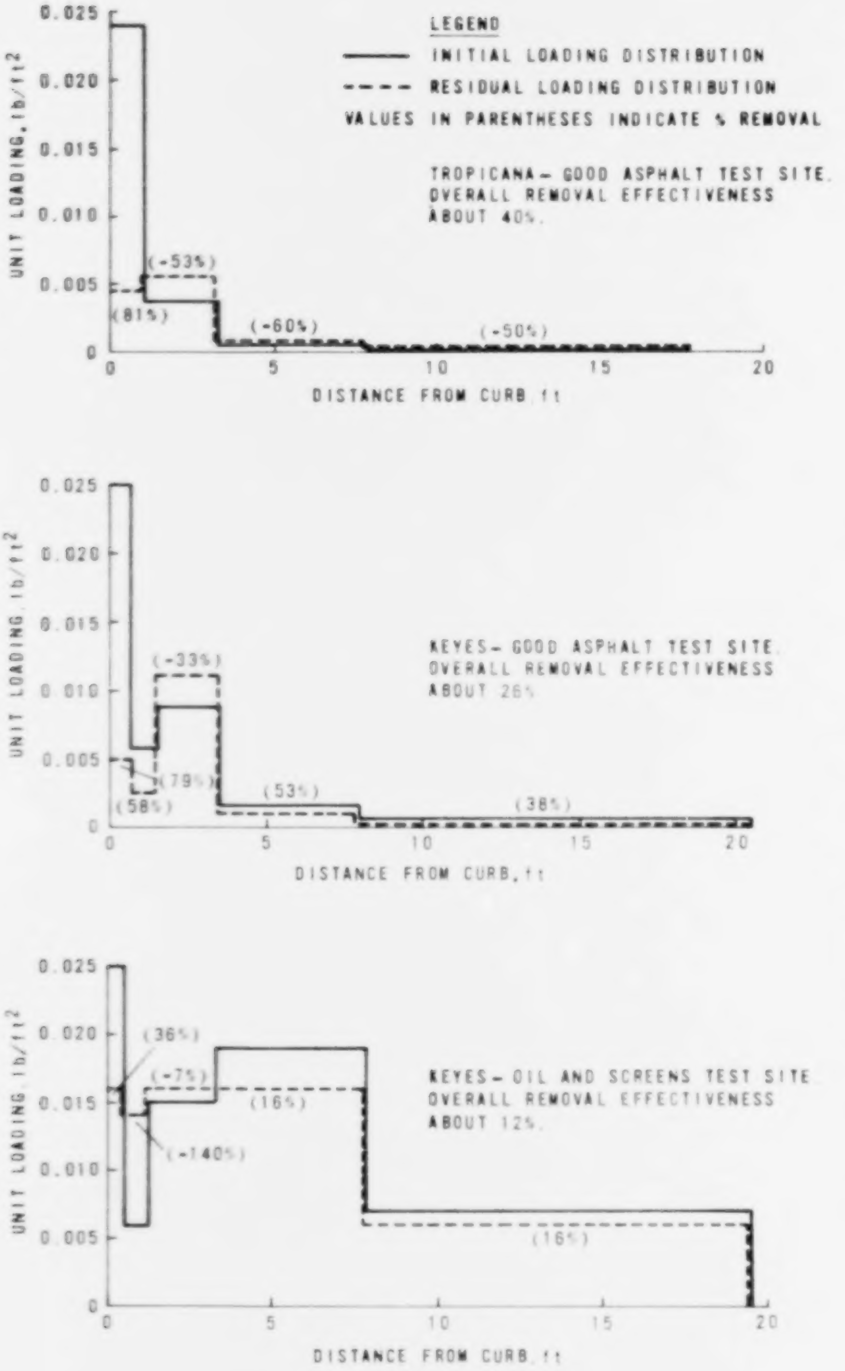


Figure 59. Redistribution of total solids from street cleaning in three different test sites, averaged for all equipment types [1].

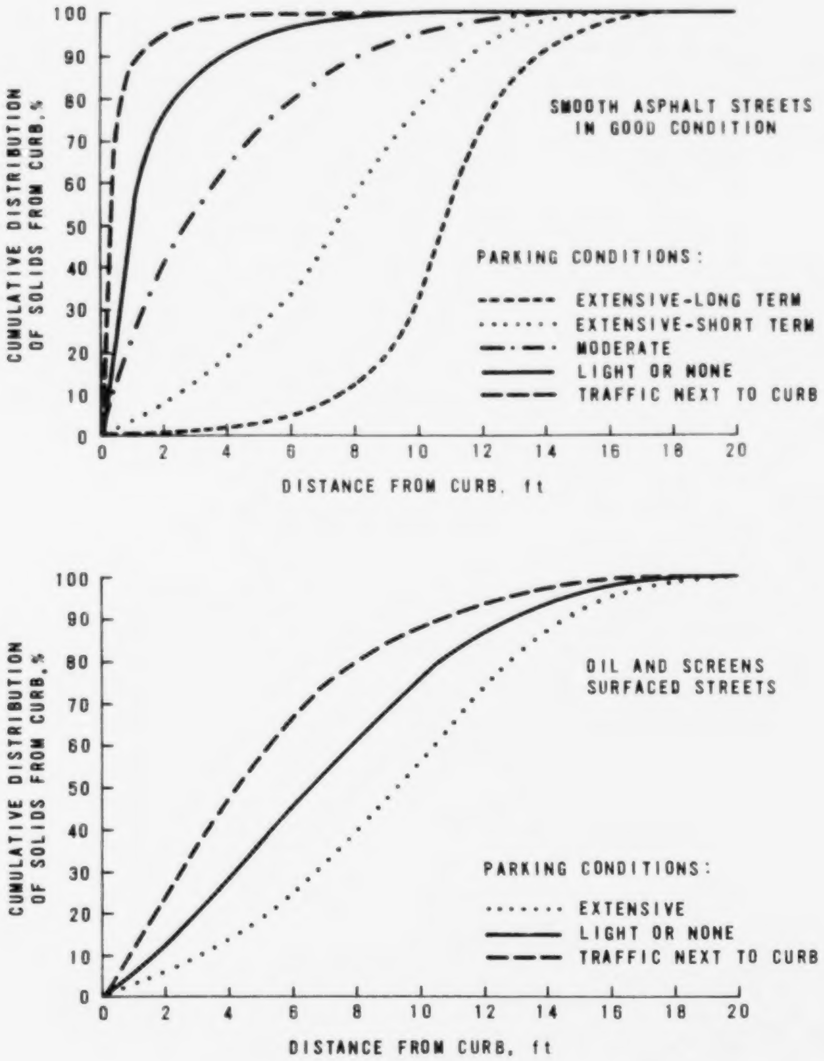


Figure 60. Effects of parking and street conditions on solids loading distribution [1].

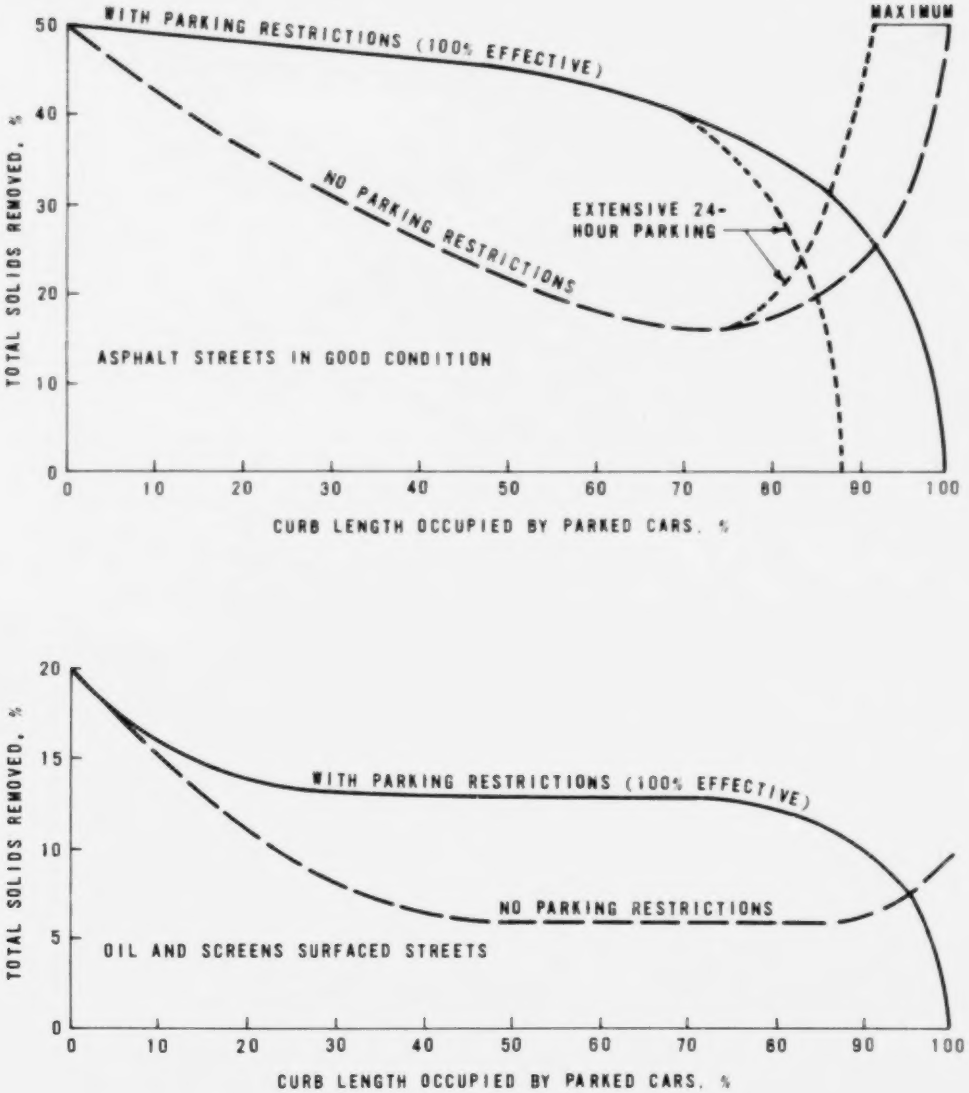


Figure 61. Effects of parking restrictions during street cleaning on solids removal from two different street surfaces [1].

## COSTS AND EFFECTIVENESS

### Costs

Extensive and detailed information on costs and labor requirements for street cleaning and all support activities in San Jose was collected and evaluated for the year ending September 30, 1977. These are summarized by specific item in Table 77.

Table 77. ANNUAL STREET CLEANING COSTS AND LABOR REQUIREMENTS, SAN JOSE [1]

| | Costs | | Labor | |
|---|---|---|---|---|
| Item | $/yr[a] | Percentage of total | Man-d/yr | Percentage of total |
| Maintenance supplies[b] | 97,000 | 12 | -- | -- |
| Operation supplies[c] | 30,000 | 3 | -- | -- |
| Debris transfer and disposal[d] | 67,000 | 8 | 780 | 13 |
| Equipment depreciation | 32,000 | 3 | -- | -- |
| Labor[e] | | | | |
| Equipment operators | 338,000 | 41 | 3,400 | 56 |
| Maintenance personnel | 183,000 | 23 | 1,200 | 20 |
| Supervisors | 83,000 | 10 | 650 | 11 |
| Total | 830,000 | 100% | 6,030 | 100% |

a. ENR 3000.

b. Includes broom replacements.

c. Tires, fuel, and oil.

d. Front-end loaders removed interim piles from streets, and dump trucks transported them (maximum 15 miles) to landfill.

e. Includes administration, warehouse, secretary, and overhead costs.

Labor accounts for about 75% of the total annual cost, making street cleaning a labor-intensive urban runoff control measure.

Those categories that might be affected by a significant change in the street cleaning equipment used (maintenance supplies and labor) make up only 35% of the total costs. Thus, a major change in equipment type could only slightly reduce the total costs.

During the same year, 89,720 km (55,761 mi) of street surface was cleaned by the San Jose Public Works Department, at $9.25/curb-km ($14.88/curb-mi) (ENR 3000). The unit labor requirement was 0.54 man-h/curb-km (0.87 man-h/curb-mi).

While the San Jose unit costs may appear high, most other evaluations do not include all the costs because few other jurisdictions have all the information available. Most other street cleaning cost evaluations include only supplies and operator labor expense (55% of San Jose's total cost), and assume the curb length cleaned equals the increase in the street cleaner's odometer reading (actually about half).

A 1973 nationwide survey of 400 city street cleaning program costs yielded typical unit costs (adjusted to ENR 3000) between \$0.026 and \$0.035/kg (\$24 and \$32/ton) removed [5]. This corresponds to about 1% of a typical city's budget, but above, did not include all the associated costs.

A large portion of the typical street cleaning budget (35% in San Jose) goes for equipment maintenance. A 1973 survey of 14 cities across the nation yielded an average total maintenance cost of \$1.62/curb-km cleaned (\$2.61/curb-mi), ENR 3000 [7]. The greatest portion of this was spent for brooms and brushes (\$0.40/curb-km) and for major repairs (\$0.39/curb-km). In comparison, San Jose's average total maintenance cost was \$3.12/curb-km (\$5.02/curb-mi).

The cost and labor requirements to remove solids increase with the number of cleaning passes per year; i.e., the rate of return decreases as the streets are cleaned more often. Computer analysis of the San Jose data revealed the relationships shown in Figures 62 and 63; unit costs increased from \$0.030 or \$0.039/kg (\$0.013 or \$0.018/lb, ENR 3000) for two passes a year to \$0.57/kg (\$0.26/lb) for 150 to 300 passes a year. More frequent cleaning lowers the solids loadings on street surfaces, and so results in lower removals per pass; the cost per pass remains about the same.

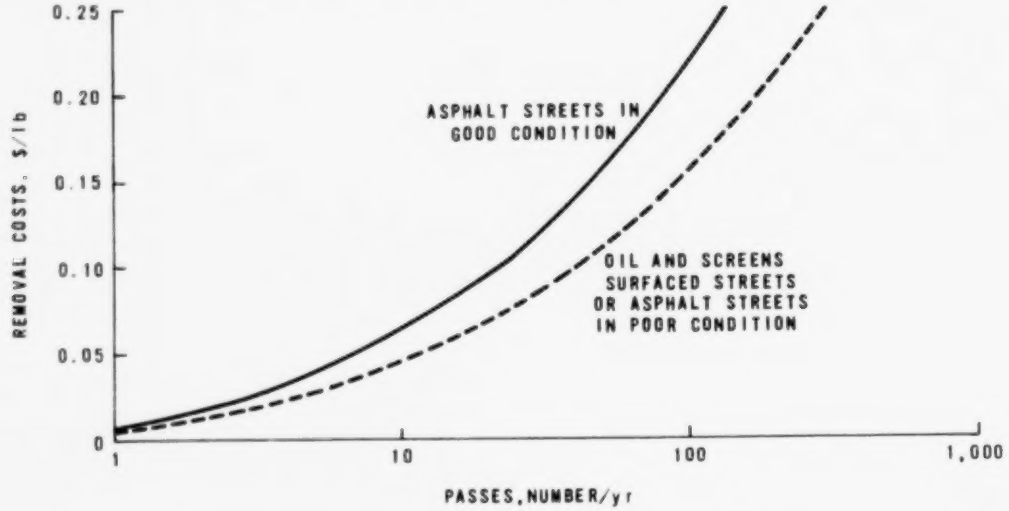


Figure 62. Variation of unit cleaning costs with number of passes [1].

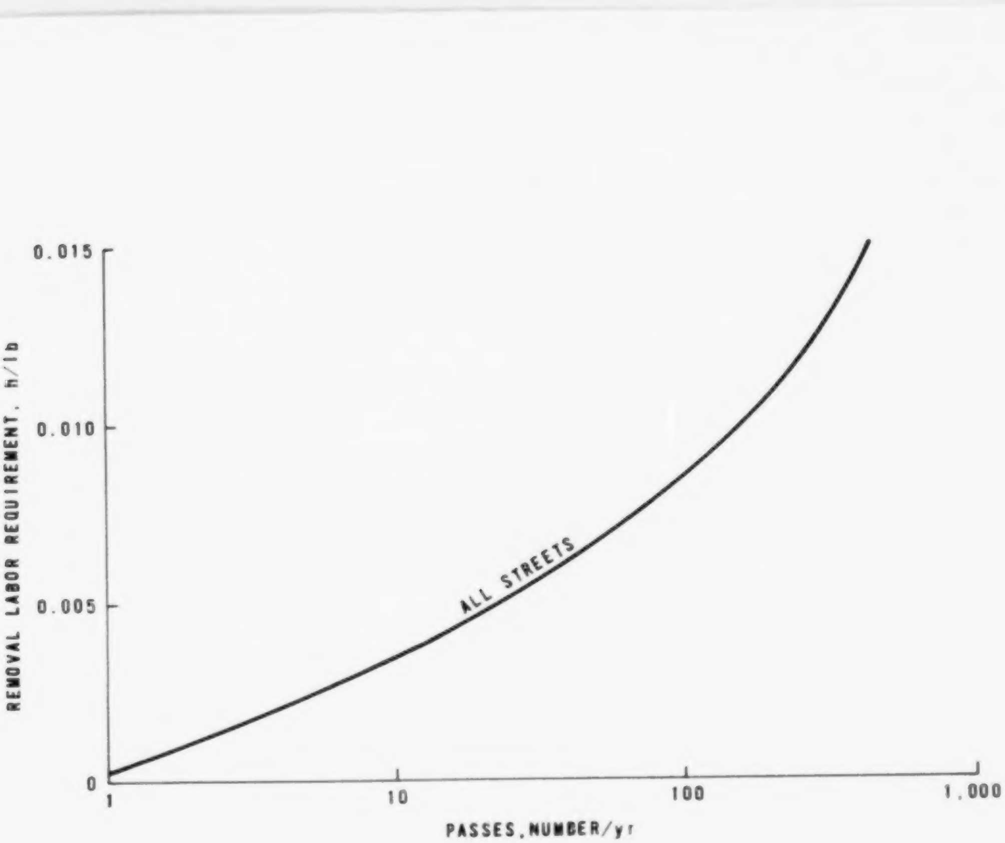


Figure 63. Variation of unit labor requirements with number of passes [1].

## Effectiveness

Street cleaning is most effective in controlling heavy metals, and moderately effective in controlling oil and grease, floating matter, and salts. It is less effective in controlling bacteria, and poorly effective in controlling sediment, nutrients, and oxygen-demanding matter. In other words, street cleaning is most effective in controlling street-originating pollutants. Any one control measure has limited capabilities, and a combination of measures is generally needed even to control a single pollutant problem. The San Jose demonstration project final report describes decision analysis methods that may be used to help select urban runoff control programs [1].

The average unit costs to remove various pollutants from the five test sites by street cleaning are presented in Table 78. Overall, the unit costs ranged from \$0.059/kg (\$0.027/lb) of total solids removed to \$133,000/kg (\$60,200/lb) of cadmium (ENR 3000). The average unit labor needs over all the test sites are also shown, and range from 0.01 h/kg (0.005 h/lb) of total solids removed to 990 h/kg (450 h/lb) of cadmium.

Comparing unit removal costs between different test areas, it costs more \$0.24 to \$0.40/kg, (\$0.11 to \$0.18/lb) to remove solids from the asphalt streets in good condition than from both the poorer quality asphalt streets \$0.06/kg (\$0.027/lb) and the oil and screens streets \$0.18/kg (\$0.08/lb).

## Table 78. COST EFFECTIVENESS FOR SAN JOSE STREET CLEANING OPERATIONS[a] [1]

|  | Keyes - oil and screens test site |  | Keyes - good asphalt test site |  | Tropicana - good asphalt test site |  | Downtown - poor asphalt condition streets test site |  | Downtown - good apshalt condition streets test site |  | All test sites |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Pollutant | Average removal, lb/curb-mi | Average unit cost, $/lb removed | Average removal, lb/curb-mi | Average unit cost, $/lb removed | Average removal, lb/curb-mi | Average unit cost, $/lb removed | Average removal, lb/curb-mi | Average unit cost, $/lb removed | Average removal, lb/curb-mi | Average unit cost, $/lb removed | Average unit labor, h/lb removed |
| Total solids | 170 | 0.08 | 130 | 0.11 | 100 | 0.15 | 540 | 0.027 | 83 | 0.18 | 0.005 |
| Suspended solids | 85 | 0.17 | 65 | 0.23 | 50 | 0.29 | 270 | 0.054 | 43 | 0.34 | 0.01 |
| Chemical oxygen demand | 12 | 1.25 | 16 | 0.91 | 9.7 | 1.45 | 61 | 0.24 | 11 | 1.35 | 0.04 |
| Biochemical oxygen demand | 6 | 2.39 | 8 | 1.87 | 4.9 | 3.01 | 31 | 0.48 | 5.5 | 2.59 | 0.08 |
| Orthophosphate | 0.0089 | 1,660 | 0.018 | 809 | 0.017 | 851 | 0.079 | 187 | 0.012 | 1,250 | 28 |
| Kjeldahl nitrogen | 2.40 | 6.02 | 0.28 | 52 | 0.21 | 70 | 1.30 | 11 | 0.60 | 91 | 2.1 |
| Lead | 0.15 | 97 | 0.81 | 18 | 0.40 | 36 | 1.00 | 15 | 0.49 | 30 | 1.1 |
| Zinc | 0.066 | 218 | 0.079 | 187 | 0.049 | 301 | 0.27 | 54 | 0.072 | 197 | 7.5 |
| Chromium | 0.071 | 208 | 0.051 | 280 | 0.039 | 374 | 0.24 | 60 | 0.047 | 311 | 9.0 |
| Copper | 0.13 | 114 | 0.081 | 176 | 0.072 | 197 | 0.50 | 29 | 0.093 | 156 | 4.5 |
| Cadmium | 0.00024 | 60,200 | 0.0003 | 48,800 | 0.00027 | 51,900 | 0.0015 | 9,650 | 0.0023 | 6,330 | 450 |

a. ENR 3000.

These comparisons are as expected, considering the different initial loadings on, nature of, and removals from the various street surfaces. Similar comparisons are generally true for the other pollutants, except in the oil and screens test site. There, street surface particulates are abundant, but the pollutant concentrations are relatively low. The major source of the particulates in this test area is street surface wear material, which is relatively "clean."

The costs of removing pollutants by street cleaning were compared with the costs of removing them from runoff by various stormwater treatment processes. When flow equalization (storage) and collection facility costs are excluded, the average unit treatment costs are significantly less than the unit street cleaning costs for all pollutants. However, when flow equalization costs are included, the average unit treatment costs are generally higher than similar costs for street cleaning; comparisons for heavy metals could not be made because of lack of data. In making such comparisons, it must be remembered that treatment measures affect only water quality, whereas street cleaning can also improve air quality, aesthetic conditions, and public safety.

## IMPACTS

### Environmental Impacts

Environmental impacts of street cleaning can include impacts on urban hydrology, receiving water quality, and air pollution.

#### Hydrology--

Without street cleaning, street litter and particulates accumulate in sewers and catchbasins when rains are sufficient to transport these materials off the streets, but not heavy enough to flush out the sewers. Street cleaning obviously reduces these accumulations, and thus helps reduce sewer plugging and the need for sewer cleaning.

#### Water Quality--

Pollutants in urban runoff were compared with their presence in the effluent from the advanced secondary sewage treatment plant serving the San Jose study areas. Occurrence ratios for those parameters in urban runoff exceeding that in sewage treatment plant effluent are presented in Table 79. These ratios suggest that urban runoff may have more important short-term impacts on receiving waters than treated sewage effluent. Lead, chromium, and suspended solids have greater annual yields in the runoff than in the treated secondary effluent. Such a tabulation helps characterize the nature of local runoff pollution problems.

The effects of street cleaning on water quality in Coyote Creek, the receiving water for runoff from the San Jose study areas, can only be inferred at this time. These effects may be more directly determined under proposed future extensions to the Coyote Creek project [2]. Much is known, however, on which to base such inferences. The nature of the pollutants in the urban runoff is known (Tables 70, 71, 72, and 79), and the extent to which street cleaning can reduce these pollutants is known (Tables 75 and 76).

### Table 79. COMPARISON OF URBAN RUNOFF AND ADVANCED SECONDARY WASTEWATER TREATMENT PLANT EFFLUENT AT SAN JOSE

| Parameter | Ratio of average runoff to STP effluent concentration | Ratio of peak (1 hr) runoff to STP effluent concentration | Ratio of runoff to STP annual yields |
|---|---|---|---|
| $BOD_5$ | 1.1 | 1.4 | 0.17 |
| COD | 5.6 | 10 | 0.20 |
| TKN | 0.28 | 1.1 | 0.005 |
| SS | 9.2 | 32 | 1.3 |
| Cd | 5 | 20 | 0.07 |
| Cr | 1.3 | 2.5 | 1.6 |
| Cu | 0.37 | 1.1 | 0.5 |
| Pb | 41 | 150 | 28 |
| Zn | 2.1 | 6.3 | 0.33 |
| Turbidity | 2.5 | 6.5 | -- |
| TOC | 3.5 | 9.7 | -- |

Pollution conditions in Coyote Creek are also known from a separate study, which involved intensive, manual sampling of the biological and water quality conditions of the creek from March through June 1978 [2]. The upper reaches of Coyote Creek pass through an undeveloped area and are not affected by urban runoff until the creek reaches San Jose. In the urbanized areas, which include the street cleaning test sites, there were no discharges other than urban runoff. The small receiving capacity of Coyote Creek is typical of many western streams.

Variations in sediment quality trends were observed along Coyote Creek as shown in Figure 64. The pollutant levels rose rapidly as the flow passed through the urbanized area of San Jose, indicating the potential water quality impacts of urban runoff can change a stream environment. The median sediment particle size decreased through the urbanized area, indicating an increased silt content. Of 12 fish species observed in the creek, only 4 occurred in the urban reaches and 98% of these were mosquito fish (*Gambusia affinis*), which are pollution tolerant. Urban samples of organism tissues contained up to three times the lead and zinc concentrations than did upstream samples. And the lower creek reaches were dominated by pollution tolerant species of benthic macroinvertebrates and attached algae.

To determine how street cleaning affects the creek would require a major change in the city's street cleaning program plus extended monitoring of creek conditions until they returned to equilibrium under the new urban runoff loads. Also, the creek pollution comes from many urban sources, and offstreet urban pollutants may not be controllable by street cleaning.

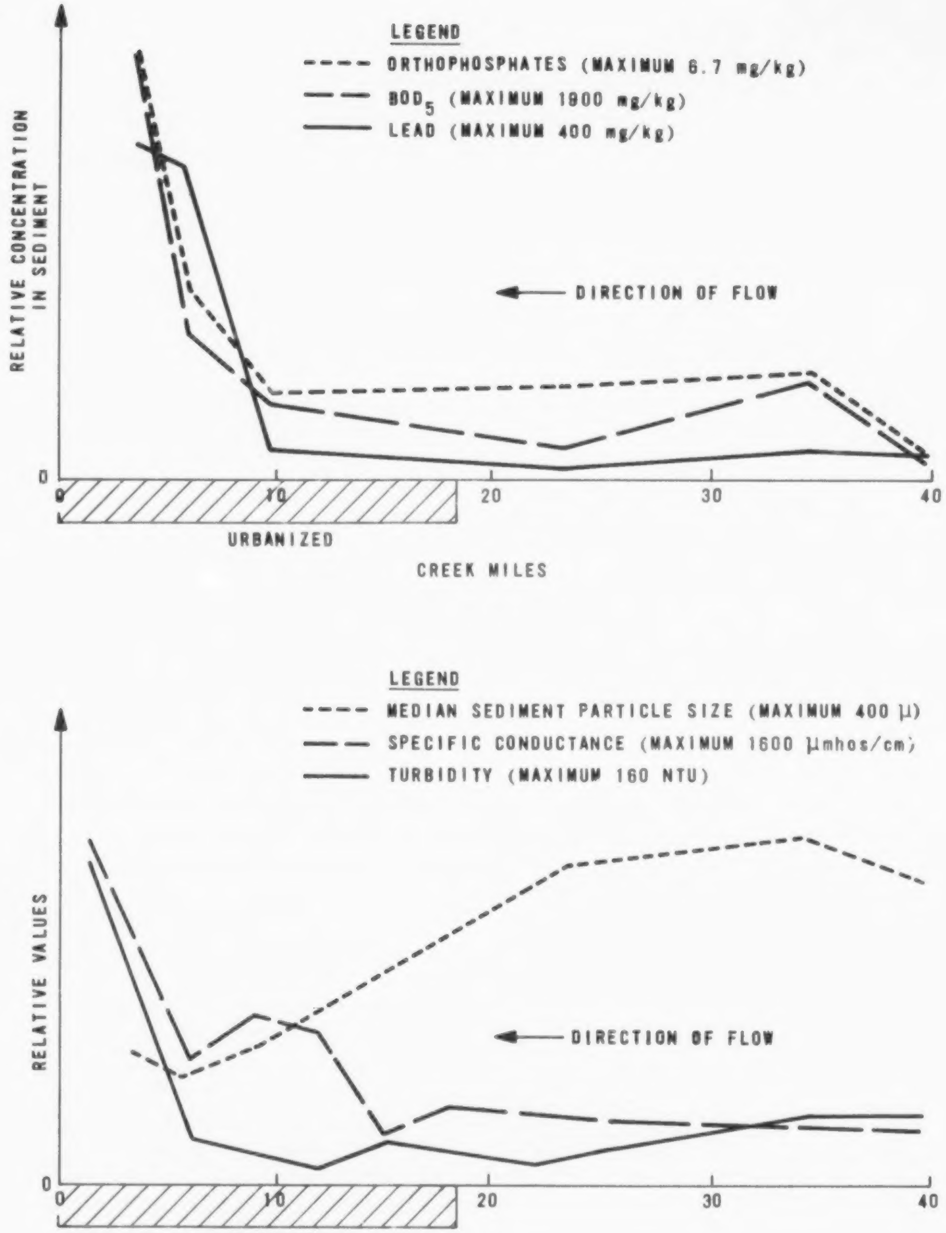


Figure 64. The variation of sediment quality along Coyote Creek [2].

Air Pollution--

From vehicle travel statistics and typical emission rates, it has been estimated that nationwide particulate emissions (<20 µ) from street surfaces totaled 32 million metric tons (35 million tons) in 1972 [1]. This source is considered very important, when compared with the estimated 1972 total of 26 million metric tons (29 million tons) of particulate emissions from all point sources combined (industrial processes, stationary fuel combustion, solid waste disposal, transportation, etc.).

The accumulation rate of particles on street surfaces is highest shortly after street cleaning or rain (Figure 55). Dust is also stirred up during street cleaning, which can immediately settle back on the streets. Over time, the accumulation rate levels off. The deposition rate is believed to be constant and approximately equal to the initial high accumulation rate, and the increasing difference between the deposition rate and the accumulation rate over time is believed to result from particulate losses to the air. In this way, dust (fugitive particulates) emissions were estimated from the accumulation rates.

A number of local factors can affect dust emissions. Those measured at the San Jose test sites, and the importance of their effects on emission rates, are listed in Table 80. Since the loading of particulates on the street surface was found to influence emissions, improved street cleaning could affect their control.

Table 80. THE IMPORTANCE OF FACTORS INFLUENCING FUGITIVE PARTICULATE EMISSION RATES

| Factor | Importance |
|---|---|
| Pavement material | High |
| Pavement condition | High |
| Particulate loading | High |
| Traffic density | High |
| Wind speed | High |
| Traffic speed | Medium |
| Particulate size distribution | Medium |
| Wind direction | Low |
| Relative humidity | Low |

About 80% of the increased number of atmospheric particulates near streets was found to occur in the 0.5 to 1.0 µ size range, whereas 90% of their weight was associated with sizes greater than 10 µ.

Emission rates for total solids from three different test sites are presented in Table 81. The rates increase considerably with time since street cleaning; therefore, street cleaning frequencies can affect fugitive particulate emission rates from road surfaces.

### Table 81. FUGITIVE PARTICULATE EMISSION RATES FOR LOSSES OF TOTAL SOLIDS FROM STREET SURFACES

| Time after street cleaning or significant rain, d | Emission rates from test sites | | | | | |
|---|---|---|---|---|---|---|
| | Tropicana - good asphalt | | Keyes - good asphalt | | Keyes - oil and screens | |
| | lb/curb-mi·d | g/vehicle-mi | lb/curb-mi·d | g/vehicle-mi | lb/curb-mi·d | g/vehicle-mi |
| 2 - 4 | 4 | 1.7 | 4 | 0.44 | <1 | <4.5 |
| 4 - 10 | 4 | 1.7 | 4 | 0.44 | 3 | 14 |
| 10 - 20 | 5 | 2.1 | 5 | 0.55 | 4 | 18 |
| 20 - 30 | 7 | 2.9 | 7 | 0.77 | 5 | 23 |
| 30 - 45 | 8 | 3.3 | 8 | 0.88 | 10 | 45 |
| 45 - 60 | 9 | 3.7 | 9 | 0.98 | -- | -- |
| 60 - 75 | 12 | 5.0 | 12 | 1.30 | -- | -- |
| Average | 6 | 2.5 | 6 | 0.66 | 4 | 18 |

Emission rates from the Keyes-good asphalt and Tropicana-good asphalt test sites were the same because their accumulation rates were similar. Major differences in traffic volumes between the three sites resulted in significantly different emission rates on the basis of vehicle travel distance. The average particulate losses from the three test sites ranged from 0.4 to 11 g/vehicle-km (0.66 to 18 g/vehicle-mi); this compares with typical range of 1.2 to 3.1 g/vehicle-km (2 to 5 g/vehicle-mi) reported by others [8, 9, 10]. The ratios of the emission rates of other pollutants to total solids are given in Table 82.

### Table 82. RELATIVE FUGITIVE PARTICULATE EMISSION RATES (BY WEIGHT) OF VARIOUS POLLUTANTS FROM THREE TEST SITES[a]

| Parameter | Keyes - good asphalt and Tropicana - good asphalt test sites | Keyes - oil and screens test sites |
|---|---|---|
| Total solids | 1 | 1 |
| Chemical oxygen demand | 0.12 | 0.05 |
| Kjeldahl nitrogen | 0.0018 | 0.00077 |
| Orthophosphates | 0.00015 | 0.00010 |
| Lead | 0.0043 | 0.0010 |
| Zinc | 0.00047 | 0.00020 |
| Chromium | 0.00033 | 0.00030 |
| Copper | 0.00052 | 0.00046 |
| Cadmium | 0.0000017 | 0.00000025 |

a. Ratio to total solids.

Such particulate losses from street surfaces can contribute a large portion of an area's total particulate emissions. The San Jose data (Table 81) suggest that cleaning an asphalt street weekly instead of every 2 or 3 months will reduce particulate emissions to one-third.

Dust levels inside the San Jose street cleaning equipment cabs and around the equipment while operating were also measured, with and without the water spray in use. Without the water spray turned on, levels in the cab were 4.6 times ambient (outside the cab), and behind the street cleaner they were 7.3 times ambient. With the water spray, the level inside the cab fell to about the same as ambient, but behind the street cleaner the level fell only 20%. Most of the concentration changes caused by the spray occurred in the smaller particle sizes.

### Socioeconomic Impacts

The high labor intensity of street cleaning should be considered a positive social impact, at least under present labor market conditions.

In the demonstration project no direct information was obtained on how street cleaning affects land values, taxes, or energy requirements.

Aesthetics and public safety obviously benefit from the litter and dust control achieved by street cleaning, regardless of its pollution abatement effectiveness. However, such impacts were not measured in the demonstration study.

## SECTION 10

## BEST MANAGEMENT PRACTICES
## MIDDLESEX COUNTY, CONNECTICUT

A number of small, onsite stormwater control facilities have been constructed in Middlesex County to reduce the impact new development has on the quality and quantity of water. Legislation at the state and local levels is oriented toward the control of erosion/sedimentation and flooding in the waterways and wetlands. Implementation of stormwater control regulations are the responsiblilty of each town, as there are no county governments.

The types of stormwater controls in Middlesex County are representative of a small systems approach for privately owned single developments. New residential and industrial development have used sedimentation ponds, recharge basins, and dry wells to achieve almost total runoff volume and pollutant load control. The feasibililty of proposed stormwater controls is evaluated on a case-by-case basis because no exact design criteria are outlined. This method allows versatility in selecting a unique solution for each site. Most facilities are conservatively designed, often controlling 10 to 25 year return frequency storms.

The adverse environmental and socioeconomic impacts of new developments are avoided to a great extent by an environmental review of the proposed development area before development approval. These studies are made, at no charge, through the local Soil Conservation District and the Eastern Connecticut Resource Conservation and Development Area [1].

### PROJECT DESCRIPTION

Urban stormwater runoff controls constructed in Middlesex County are representative of small-scale facilities used for single and often isolated new developments. The purpose of these controls is to prevent flooding, erosion, and water quality problems in the major rivers and streams. Regulatory requirements are enforced at the local level through land use restrictions and ordinances that require developments to provide facilities for handling increased runoff from the developed area.

Three typical, private runoff control installations, two industrial sites and one residential site, are used as examples of the small-scale facilties. Since most facilities are privately owned, no monitoring of efficiency, particularly pollutant reduction efficiency, has been conducted. Rather, these facilities illustrate the type of controls that have been used and appear to be a most promising solution for controlling runoff.

## Area Characteristics

Connecticut is in the southwestern portion of New England and is bordered on the south by Long Island Sound. Middlesex County is in the south central portion of the state as shown in Figure 65.

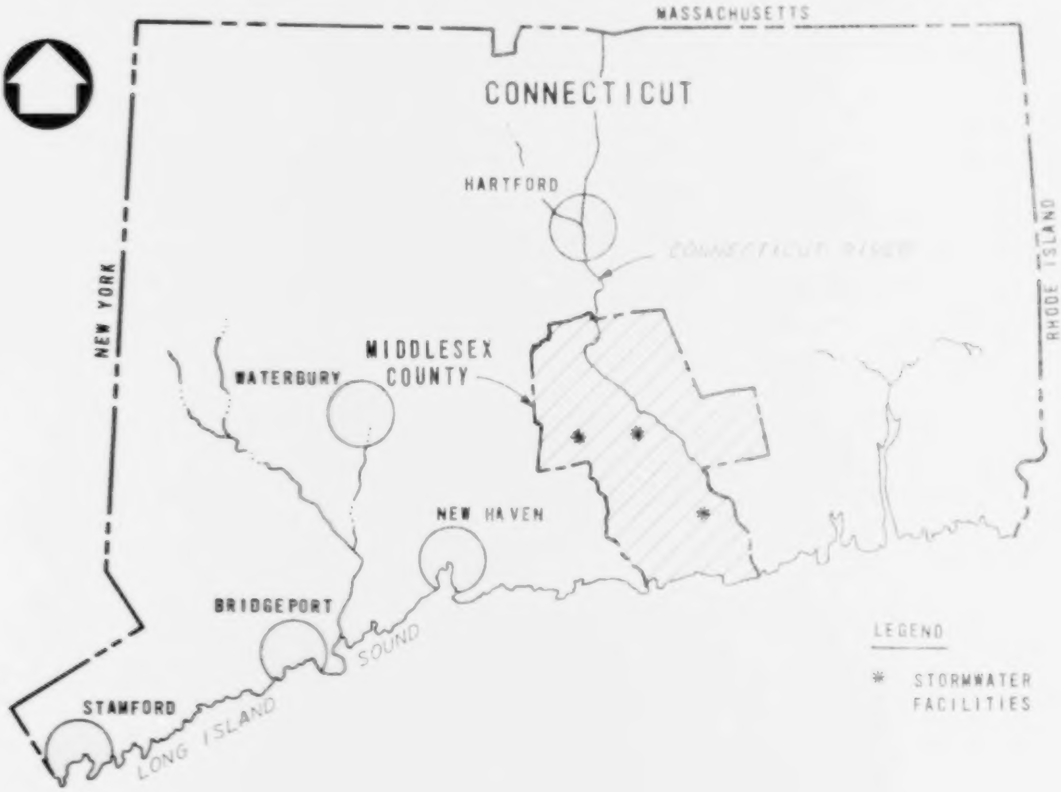


Figure 65. Middlesex County, Connecticut.

The topography of Connecticut is predominantly rolling hills, varying in elevation from 90 to 600 m (300 to 2,000 ft); the highest elevations in Middlesex County are near 300 m (1,000 ft). The state is bisected by the Connecticut River and has numerous rivers and streams in four other major river basins in the state.

The annual precipitation in Middlesex County is about 117 cm (46 in.), which falls in relatively uniform amounts throughout the seasons. Heavy rainfall storms occur during the summer and fall, usually resulting from localized thunderstorms that occur on the average of about 20 to 30 days per year. During the spring, melting snow, combined with rainfall, produces high runoff volumes and is one of the principal causes of flooding.

Connecticut has five major population centers but is mostly rural. About two-thirds of the state is forest with the remaining area developed; about 20% of the developed area is agricultural. Middlesex County, just south of Hartford, is rural with many small towns that have light industrial developments.

## Problem Assessment

The State of Connecticut has become aware of the erosion, sediment deposition, and water quality problems associated with residential and industrial development. The increased degradation of the rivers and streams by sediment deposition has been determined to be the most critical problem in Connecticut [1]. Many areas in the state have soils with relatively large percentages of fine-grained particles of clay and silt. These particles remain in suspension longer because of their lower settling velocities, which prolong turbidity. Pollutant characteristics of urban runoff have not been monitored; however, runoff quality is assumed equivalent to representative nationwide values for similar land uses.

## Approach to Stormwater Control

Middlesex County has numerous stormwater control facilities that were constructed as a result of erosion and sediment control ordinances. The State of Connecticut passed legislation to provide local jurisdictions with authority to regulate and enforce stormwater ordinances. The local ordinances require new developments to control stormwater both during and after construction.

During construction, basic guidelines of the stormwater ordinances provide for erosion and sedimentation control and apply to all changes of the land surface, including clearing, grading, excavation, and filling. The ordinances state that the development should be suited to the site, topography, and soils to minimize erosion potential; retain and protect native vegetation; and minimize land exposure. The development should also use soil and surface stabilization techniques to protect exposed areas (temporary vegetation or mulching, or both) and have sediment or desilting basins to collect runoff [2].

Postdevelopment controls required by the ordinances must accommodate the increased runoff caused by changes in soil and surface conditions (imperviousness), erosion, flooding, and sediment damage to the development property, adjacent property, or receiving waters. No specific design requirement is stated for the control facilities except for stormwater conveyance systems. All culverts and other stormwater drainage systems, except street drainage, are designed for a 25 year return frequency. Street drainage systems are designed on a 10 year return frequency, or as approved according to the condition of the site.

The control program prevents alteration of natural water courses, natural drainage and runoff, or existing drainage and runoff by controlling the discharges into storm drains, ditches, streams, or rivers with adequate carrying capacity for the additional runoff volume [3]. The ordinance provisions generally allow wide enough latitude in the types and degree of control to fit each development site. Final stormwater plans have to be approved before development is allowed to proceed, and most facilities have been conservatively designed.

## Implemented Facilities

Three facilities were selected that demonstrate a most promising technology in controlling stormwater and are representative of the types of control used in Middlesex County. Two of the controls serve light industrial developments and the third, a residential development.

Most of the facilities incorporated both pre- and post-development measures to control erosion, sedimentation, and runoff volume. Two of the facilities control all of the onsite runoff.

### Sedimentation Ponds (Industrial Site)--

A light industrial site has been developed in an environmentally sensitive area in the town of Essex. The dual use of the area for industry and open space to preserve the surrounding wetlands required substantial mitigating measures by the developers.

The Essex industrial site is adjacent to the Mud River on approximately 20 ha (50 acres) of low lying wetlands. The Mud River, classified as "A" Standard (the cleanest rating, suitable for water supply), has an estimated flow range from about 0.03 to over 0.6 $m^3/s$ (1 to over 20 $ft^3/s$), and drains an area of about 830 ha (2,050 acres) [4]. The river basin is identified as a future permanent open space with limited development and as a high priority aquifer area. The Connecticut River Estuary Regional Planning Agency (CRERPA) recommended in its Regional Plan of Development that the area be protected as a natural resource [4].

The developers of the industrial site had to provide erosion and sedimentation controls during construction and stormwater controls as permanent facilities. The final runoff control facilities, constructed in a two phase program, are shown in Figure 66.

The first phase involved constructing a primary sedimentation pond to treat runoff from the area during construction. Some portions of the site could not drain into this pond and the runoff from these areas was controlled by a barrier of hay bales between the construction site and the Mud River. The second phase involved constructing the facilities to control runoff principally from the impervious areas of the development.

Approximately 40% of the developed site, building and parking area, is now impervious. Stormwater is collected from the roof of the building and the parking area and is piped to the primary sedimentation pond. It then flows to the secondary sedimentation pond, which also receives flow from two intermittent streams. Flows are then conveyed to the marsh control area, then discharged to the Mud River. Stormwater collected at the northern end of the building, a new extension, is piped to a separate pond that discharges to the Mud River. The stormwater facilities and service area are shown in Figure 67.

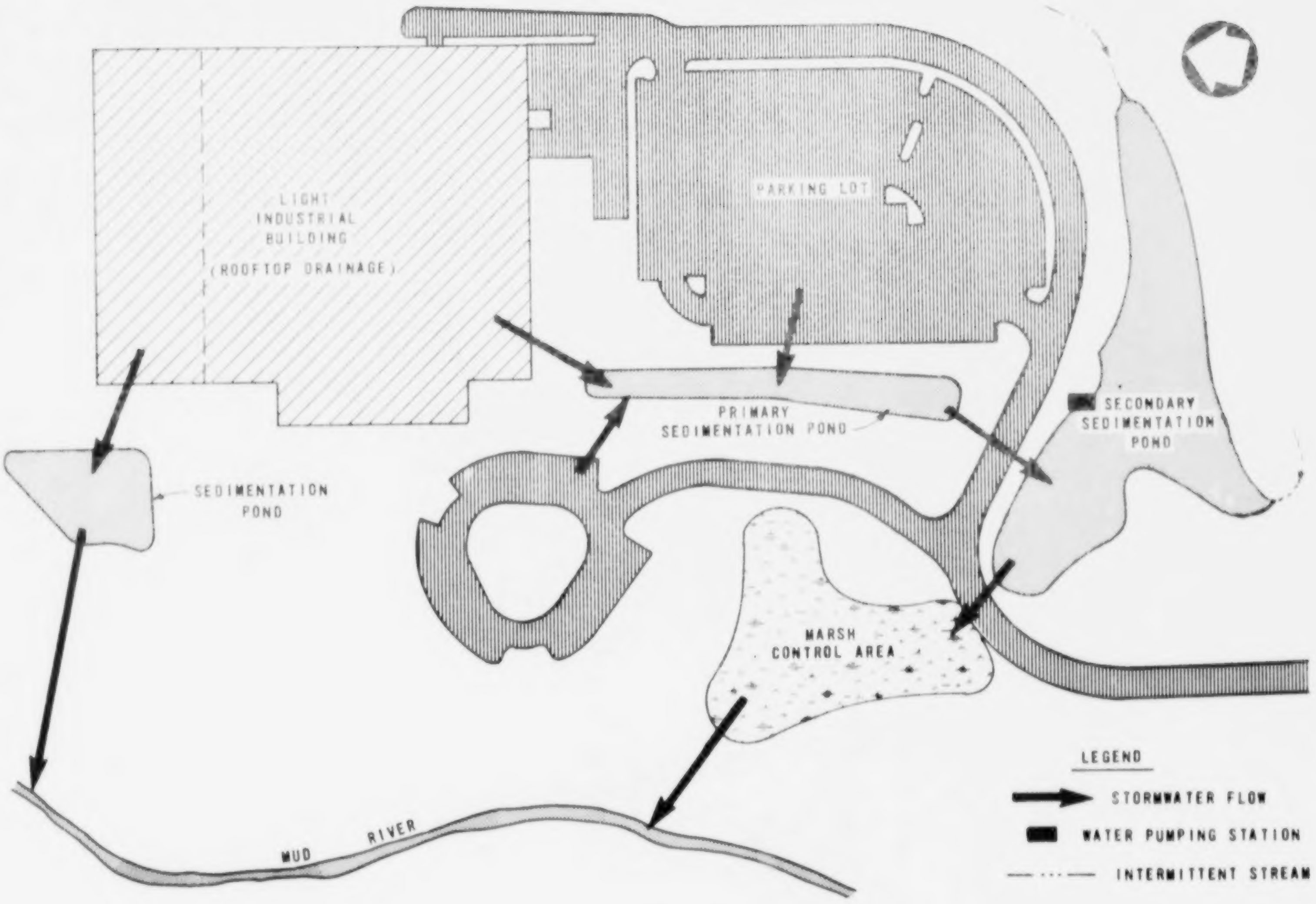


Figure 66. Schematic of stormwater control at an industrial site, Middlesex County.

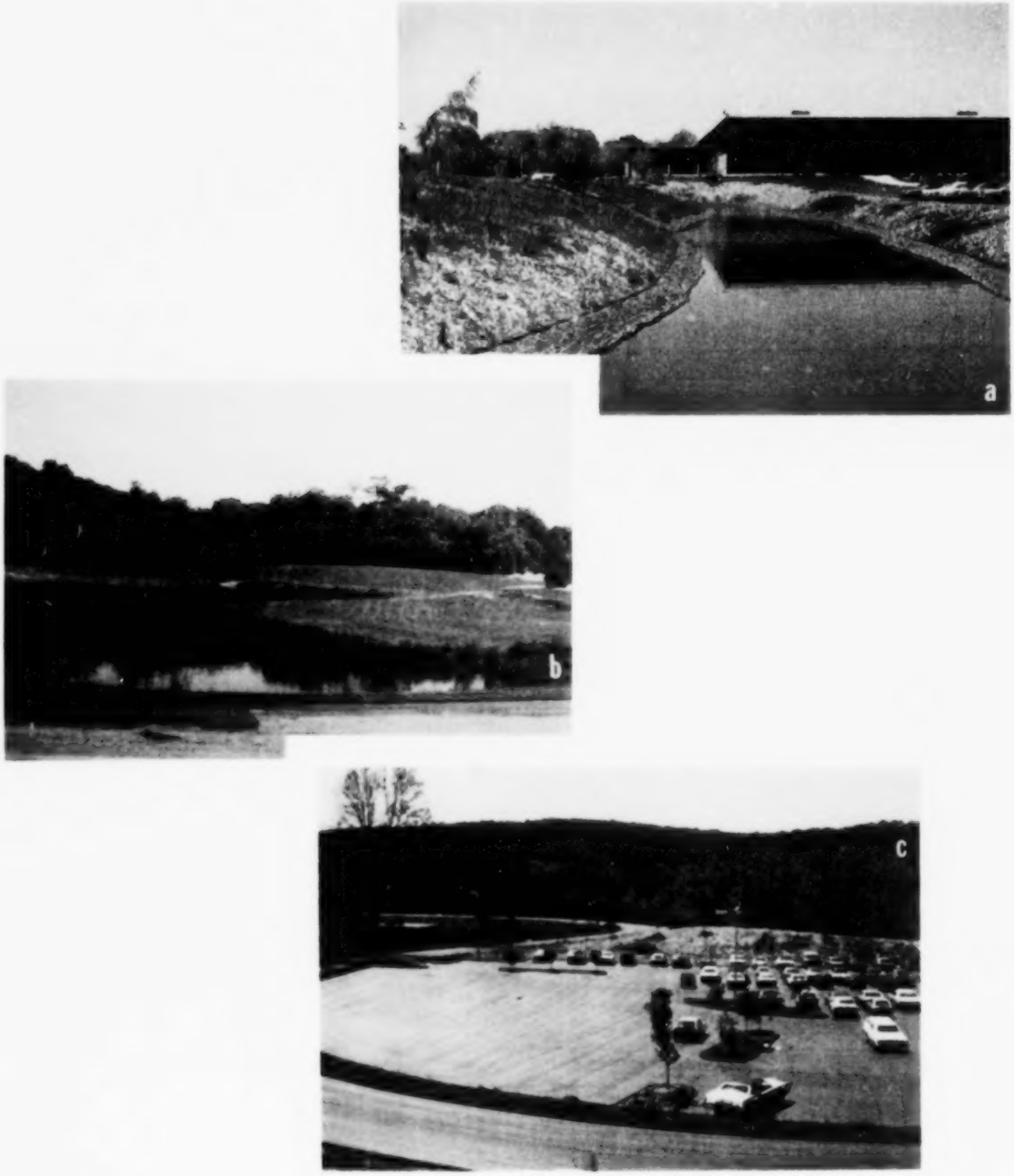


Figure 67. Stormwater control facilities at an industrial park, Middlesex County: (a) primary sedimentation pond, (b) secondary sedimentation pond, and (c) a portion of the parking area served by the sedimentation ponds.

The facilities were designed to handle a 50 year, 24 hour rainstorm. Runoff calculations were based on the SCS methodology [5]. The secondary sedimentation pond incorporated additional storage capacity and a pumping station for fire-fighting capabilities for the building. The design criteria for the stormwater control facilities are presented in Table 83.

Table 83. DESIGN PARAMETERS FOR INDUSTRIAL STORMWATER FACILITIES IN ESSEX

| Parameter | Value |
|---|---|
| Design storm | |
| Design storm frequency, yr | 50 |
| Design storm duration, h | 24 |
| Design rainfall, in. | 6.4 |
| Contributing area, acre | 43 |
| Average watershed slope, % | 6.5 |
| Primary sedimentation pond | |
| Pool surface area, $ft^2$ | 17,500 |
| Length, ft | 350 |
| Width, ft | 50 |
| Average depth, ft | 5 |
| Total volume, $ft^3$ | 88,000 |
| Permanent pool volume, $ft^3$ | 23,000 |
| Stormwater storage volume, $ft^3$ | 65,000 |
| Peak discharge rate, $ft^3/s$ | 22 |
| Detention time at peak discharge rate, h | 2.0 |
| Secondary sedimentation pond | |
| Pool surface area, $ft^2$ | 65,000 |
| Average depth, ft | 5.5 |
| Total volume, $ft^3$ | 500,000 |
| Permanent pool volume, $ft^3$ | 350,000 |
| Stormwater storage volume, $ft^3$ | 150,000 |
| Peak discharge rate $ft^3/s$ | 34 |
| Detention time at peak discharge rate, h | 3.2 |
| Marsh control area | |
| Surface area, $ft^2$ | 30,000 |
| Peak inflow rate, $ft^3/s$ | 34 |

### Dry Wells and Drainage Swales (Industrial Site)--

A light industrial site, developed in the town of Durham, is located in an area of well drained to excessively drained soils with a sandy and gravelly substratum on rock terraces. The area has slightly rolling hills and is heavily forested with considerable ground cover. No streams of major significance are near the industrial site.

The stormwater control facilities serve a total area of 3 ha (7.5 acres), of which 1,860 $m^2$ (20,000 $ft^2$) is building, 2,830 $m^2$ (30,500 $ft^2$) is parking lot, 0.8 ha (2 acres) is woods, and the remaining area of 1.8 ha (4.35 acres) is lawn.

The site and the permanent stormwater control facilities are shown in Figure 68. Two phases of stormwater control were implemented to meet the Durham requirements for erosion control during construction and postconstruction runoff control. Erosion was controlled by a temporary retaining berm on the western property line with release openings every 60 m (200 ft). The openings had riprap and hay bales to filter the sediment in runoff. Permanent postconstruction control facilities use the well-drained soils as an integral feature in operating the controls. A large drainage swale collects runoff from the lawn area, providing percolation and evaporation. Percolation dry wells collect and control stormwater runoff from the parking lot, building roof, and some runoff from the lawn areas.

The design of the facilities was based on the soil's percolation capacity. Soil borings indicate topsoil depths of about 0.23 m (0.75 ft) on top of sand and sandy till with average percolation rates of 2.54 cm/8 min (1 in./8 min). The dry wells were installed in two groups of three and four in series, together providing about 36 $m^3$ (1,275 $ft^3$) of storage capacity with nearly 19 $m^2$ (200 $ft^2$) of percolation surface.

### Recharge Basin (Residential Site)--

A small residential site, which was a sand and gravel pit, has been developed adjacent to the Connecticut River in the town of Haddam. A stormwater recharge basin provides runoff control for the site and adjacent areas by using the high percolation rates of the soils and storage from previous excavations. Land reclamation, control of stormwater runoff, and the resulting recharge of the groundwater are positive environmental impacts of the project. The recharge basin was selected over the alternative of providing a drainage system because a drainage system would require easements through private property to discharge flow into the Connecticut River.

The residential site is on 5.5 ha (13.5 acres) of reclaimed land. The land is gently rolling and is heavily forested. The developed site is flat and the soil is generally classified by SCS as group "A" (low runoff potential). The soil consists of deep, well to excessively drained sands and gravels, and has a high infiltration rate, better than 2.54 cm/4 min (1 in./4 min), when thoroughly wetted.

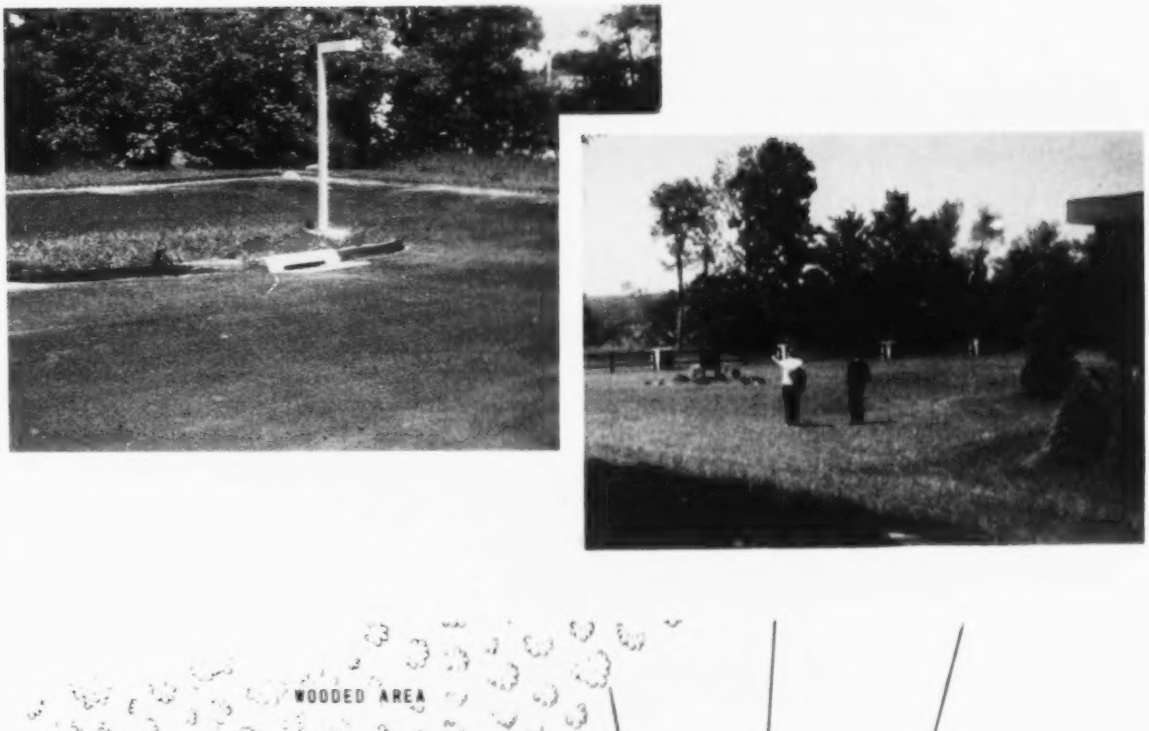


Figure 68. Industrial stormwater control using percolation, Middlesex County: (a) depressed parking lot with catchbasin inlet, (b) grassed drainage swale, and (c) schematic of the industrial site and the stormwater control facilities.

The site is divided into 13 parcels of approximately 0.4 ha (1 acre) each, one of which is the recharge basin. Runoff is collected and transported to the recharge basin by about 900 m (3,000 ft) of grassed drainage swales, which are riprapped in those portions subject to erosion. The 0.4 ha (1 acre) recharge basin has a 1.5 m (5 ft) chain-link fence around its perimeter. It is seeded with a native grass to prevent erosion. The site and its stormwater facilities are shown in Figure 69.

The design of the recharge basin assumed that the development site land use would be single-family residential and would include the runoff contribution from an additional 9.7 ha (24 acres) of land adjacent to the development site. The recharge basin contains and percolates all the flow from the total area for a 100 year storm. The design criteria for the recharge basin are presented in Table 84.

Table 84. DESIGN PARAMETERS FOR RESIDENTIAL STORMWATER FACILITIES IN HADDAM

| Parameter | Value |
|---|---|
| Design storm | |
| Design storm frequency, yr | 100 |
| Design storm duration, h | 24 |
| Rainfall, in. | 7.1 |
| Contributing area, acre | 37.5 |
| Average watershed slope, % | |
| Developed site (13.5 acre) | 4 |
| Adjacent contributing area (24 acre) | 23 |
| Recharge basin | |
| Surface area, ft$^2$ | 44,400 |
| Length, ft | 270 |
| Width, ft | 180 |
| Depth, ft | 17 |
| Capacity, ft$^3$ | 457,000 |
| Side slopes | 3:1 |
| Percolation rate, in./min | 1 |
| Percolation area, ft$^2$ | 44,400 |

## PERFORMANCE

Information on the performance of the three small, onsite stormwater control facilities is limited. The facilities have operated for about three years, but because of their rural locations and private ownership, no monitoring has occurred. Projections of the pollutant loadings and runoff volume reductions were used to estimate the level of efficiency expected for these facilities. The projections are based on representative pollutant concentrations for the various land uses that are assumed consistent with conditions at the development sites. These concentrations are summarized in Table 85.

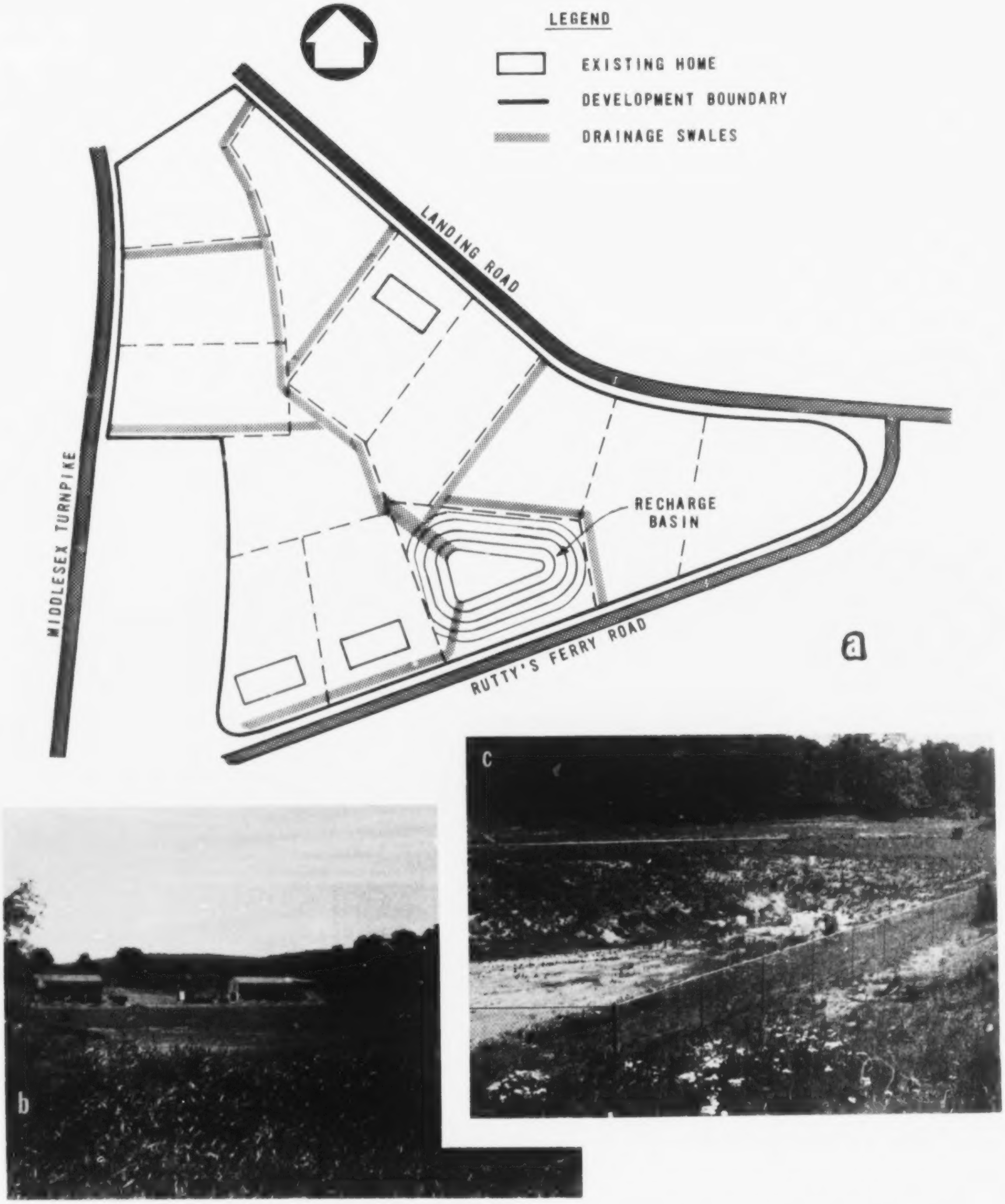


Figure 69. Residential development with stormwater control facilities, Middlesex County: (a) schematic of the recharge basin and service area, (b) view of reclaimed area and homes under construction, and (c) the stormwater recharge basin.

Table 85. REPRESENTATIVE POLLUTANT CONCENTRATIONS IN STORMWATER RUNOFF FOR SEVERAL LAND USES [6-8]

| Land use | $BOD_5$, mg/L | Suspended solids, mg/L |
|---|---|---|
| Rooftop drainage | 10 | 30 |
| Paved parking lot | 20 | 320 |
| Low density residential | 20 | 280 |
| General forest | 20 | 66 |

### Volume Reduction

The runoff volume has been reduced 100% by the residential recharge basin and the industrial dry wells, drainage swales, and percolation facilities because of the high percolation rates of the soils at these particular sites. Assuming that the dry wells could provide the reported percolation rates throughout a storm, calculations show a 100% volume reduction for a 10 year, 24 hour storm. During the 3 years of operation, the dry wells have contained the flow from all storms. The residential recharge basin, designed to contain the total volume of runoff for a 100 year storm, would be 100% effective for any storm less than the design storm regardless of the time required to percolate the stormwater.

The sedimentation ponds at the other industrial site were not designed to reduce the total volume of runoff, but rather to detain the runoff before it is discharged to the receiving water. The ponds have a combined stormwater storage volume capacity of about 6,100 $m^3$ (215,000 $ft^3$) and provide a total estimated detention time, at peak storm flow, of about 5 hours.

### Pollutant Loading Reduction

The reduction of pollutant loadings by the stormwater facilities providing storage and using percolation methods are high and are estimated to have reduced pollutant loadings, suspended solids and $BOD_5$, to the stream by 100%. The site using sedimentation ponds to discharge stormwater to a marsh area before it enters the river has an estimated reduction of 95% for suspended solids. The reduction of $BOD_5$ in the sedimentation ponds has not been measured or estimated, but is assumed high. The estimated performance of all three facilities is summarized in Table 86.

## COSTS

Most of the stormwater controls in Middlesex County are required of and provided by private developers who also bear the construction and the operation and maintenance costs of the facilities. Most of the costs of the controls is for construction, over 50% of which is associated with excavation and grading. Annual operation and maintenance costs are usually minimal and are generally limited to landscape upkeep.

### Table 86. ESTIMATED ANNUAL POLLUTANT LOADING REDUCTIONS BY STORMWATER CONTROL FACILITIES, MIDDLESEX COUNTY

| Facility | Loadings, lb/acre·yr | | Pollutant reduction, % | |
|---|---|---|---|---|
| | SS | $BOD_5$ | SS | $BOD_5$ |
| Sedimentation ponds (Industrial site) | 1,850 | 150 | 95 | -- |
| Dry wells (Industrial site) | 2,060 | 160 | 100 | 100 |
| Recharge basin (Residential site) | 550 | 70 | 100 | 100 |

No construction costs for the three example facilities were available; therefore, estimates of the capital costs were based on estimates of the quantity of excavation, materials, and equipment, using standard construction cost estimating guides. These costs are assumed representative of small-scale onsite stormwater controls, usually serving less than 4 ha (10 acres) of impervious area.

Capital cost estimates for the stormwater controls are shown in Table 87. The construction costs can vary significantly, depending on the type of control used and the availability and use of natural land features integrated in the control design. Unit costs range from about $7,500 to $16,000/ha ($3,000 to $6,600/acre) of impervious area.

### Table 87. SUMMARY OF COST ESTIMATES OF STORMWATER CONTROLS, MIDDLESEX COUNTY[a]

| Facility | Impervious area served, acre | Capital cost, $ | Cost/area, $/acre[b] | Treatment or volume reduction cost, $/Mgal | Cost of pollutant removal $/acre·lb·yr[b] | |
|---|---|---|---|---|---|---|
| | | | | | SS | $BOD_5$ |
| Sedimentation ponds | 6.53 | 29,000 | 4,400 | 7,000[c] | 0.40 | -- |
| Dry wells | 1.15 | 7,600 | 6,600 | 6,900[d] | 2.80 | 35.70 |
| Recharge basin[e] | 5.2 | 15,700 | 3,000 | 420[d] | 0.20 | 1.10 |

a. ENR 3000.

b. Based on impervious area.

c. Treatment cost for design storm runoff.

d. Volume reduction cost for design storm runoff.

e. Residential area, pollutant loadings based on total area.

The operation and maintenance costs are usually minimal because the facilities are self-activating and require little care. Several of the controls are used as recreational areas, and landscape maintenance is provided as a part of the overall grounds upkeep. After construction of the Essex industrial complex, the primary sedimentation pond required dredging to remove the sediment accumulated during construction before it was converted to a postconstruction control. The cost of dredging was about $500.

## IMPACTS

The environmental and socioeconomic impacts of the onsite stormwater control facilities in Middlesex County are highlighted by the use of small, almost totally effective control measures that maximize the use of existing physical conditions of the land. The facilities are integrated into the surrounding landscape and protect the environmentally sensitive areas.

### Environmental Impacts

The stormwater controls used in Middlesex County protect the surrounding environment from land use changes, increased impervious areas, and land disturbances caused by development. Several of the industrial developments were constructed in wetlands areas and would not have been allowed unless they adequately controlled runoff.

Each proposed development must provide runoff control plans and receive an environmental review assessment to ensure that it protects and is compatible with potential environmentally sensitive areas. The assessment evaluates the topography, geology, soils, natural drainage, aquifers, and groundwater conditions. Natural vegetation and ground cover and their ecological function are also assessed. Developments in wetlands areas are designed to be compatible with and to maintain the condition and function of the wetlands. Other impacts, such as air pollution, erosion and sedimentation, and flooding, are also assessed.

The three example stormwater control facilities were conservatively designed and provide almost total runoff pollutant and volume control. Collectively, they prevent an estimated 18,200 kg/yr (40,000 lb/yr) of suspended solids and about 1,700 kg/yr (3,700 lb/yr) of BOD from entering receiving waters. The sites control a total development area of about 18 ha (45 acres) and, on the average, reduce suspended solids by about 1,000 kg/ha·yr (890 lb/acre·yr). These unit removals represent a significant pollutant load potential that could significantly affect wildlife habitats and supporting vegetation and the characteristics and ecology of the receiving streams.

Preservation of the natural environment is one of the principal considerations in the design of new developments. The Essex industrial development in a wetlands area provides a wildlife area around the sedimentation ponds. The area was planted with a mix of trees, grasses, small grains, shrubs, and vines as shown in Figure 70.

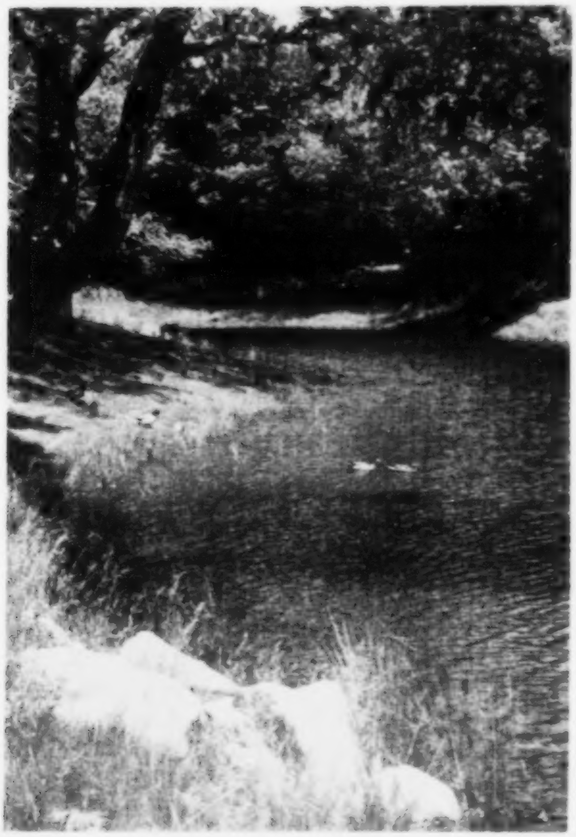
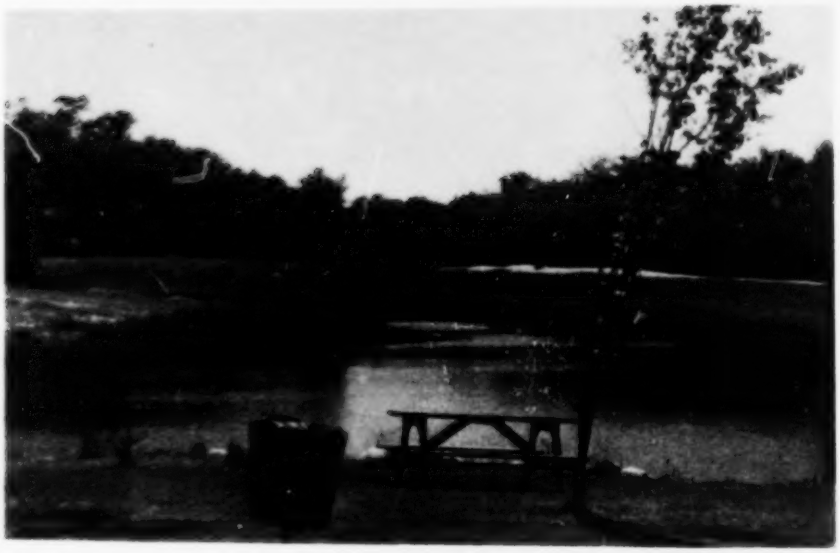


Figure 70. Vegetation around sedimentation pond supporting wildlife displaced by the adjacent industrial site, Middlesex County.

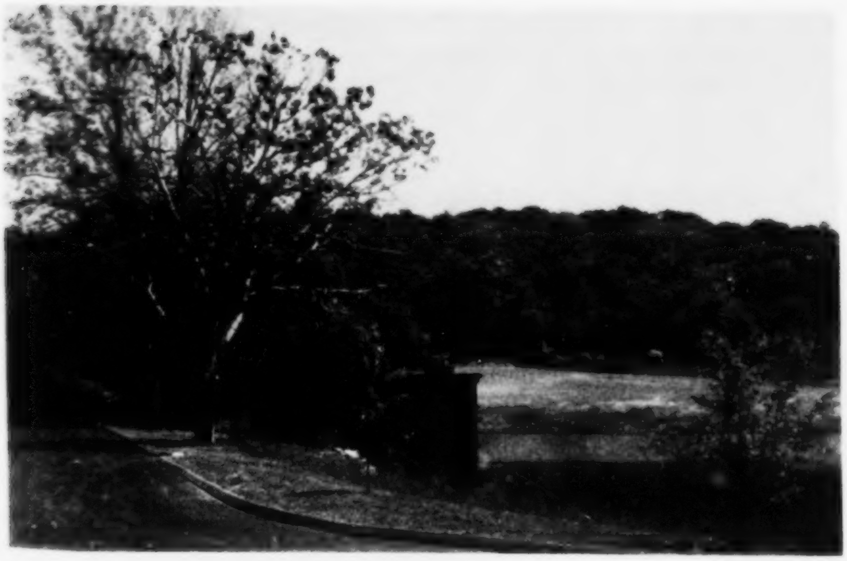
## Socioeconomic Impacts

The socioeconomic impacts of the stormwater controls have been minor, principally because the controls are small onsite facilities constructed on private developments. The developer is responsible for the cost of construction; therefore, the general public is not burdened with costs, either through assessments or from local city budgets.

Several developments have incorporated stormwater controls into the overall landscaping as shown in Figure 71. The sedimentation ponds have permanent pools, and the adjacent landscaped areas are used for recreation and picnics and improve aesthetics. The sedimentation pond also has a multipurpose use with extra pool capacity and a pumping station for fire-fighting capability.



(a)



(b)

Figure 71. Landscaped multiuse stormwater detention pond, Middlesex County: (a) picnic facilities for employees, and (b) fire-fighting water source and pumping station for the industrial complex.

## SECTION 11

## STORMWATER RUNOFF CONTROL
## BOULDER, COLORADO

The primary stormwater concern in Boulder, Colorado, is controlling the flowrate of stormwater and snowmelt runoff. The city is located in the foothills of the Rocky Mountains and is susceptible to flooding. Land development near the natural water courses that flow through the city and on the steeper slopes of the foothills has increased the potential for runoff, flooding, and erosion/sediment deposition. The effect of stormwater and snowmelt runoff on the long-term water quality of the receiving streams in Boulder is currently unknown.

Boulder has established a program to control stormwater runoff. The principal BMPs are the construction or enlargement of drainageways to accommodate the flow from a 100 year storm; onsite source detention; and land use planning. Instream detention ponds are used with new or enlarged drainageways, and sedimentation ponds are built downstream of residential developments on steep hillsides to reduce suspended solids in runoff.

### PROJECT DESCRIPTION

The stormwater and snowmelt runoff problems in Boulder are representative of the problems in semiarid, mountainous areas where the quality of the receiving streams near the smaller population centers is high. As development increases, however, the effect of increased runoff has created flooding and the potential for water quality problems. In Boulder, water quality is currently considered a secondary problem to flooding, but limited monitoring shows that the effects of stormwater pollutants may become a more important issue.

### Area Characteristics

Boulder is northwest of Denver in the foothills of the Rocky Mountains. The city is about 32 km (20 mi) east of the Continental Divide. The elevations in Boulder vary from about 1,600 to 1,900 m (5,300 to 6,200 ft) and generally slope from west to east [1]. The location of Boulder is shown in Figure 72.

Boulder's annual temperatures range between -9° and 30.5°C (16° and 87°F) with an annual precipitation of about 47 cm (18.5 in.). Precipitation is light in the winter, 16% of the annual precipitation. About 52% of the annual precipitation occurs between March and June, creating large spring runoff volumes [2].

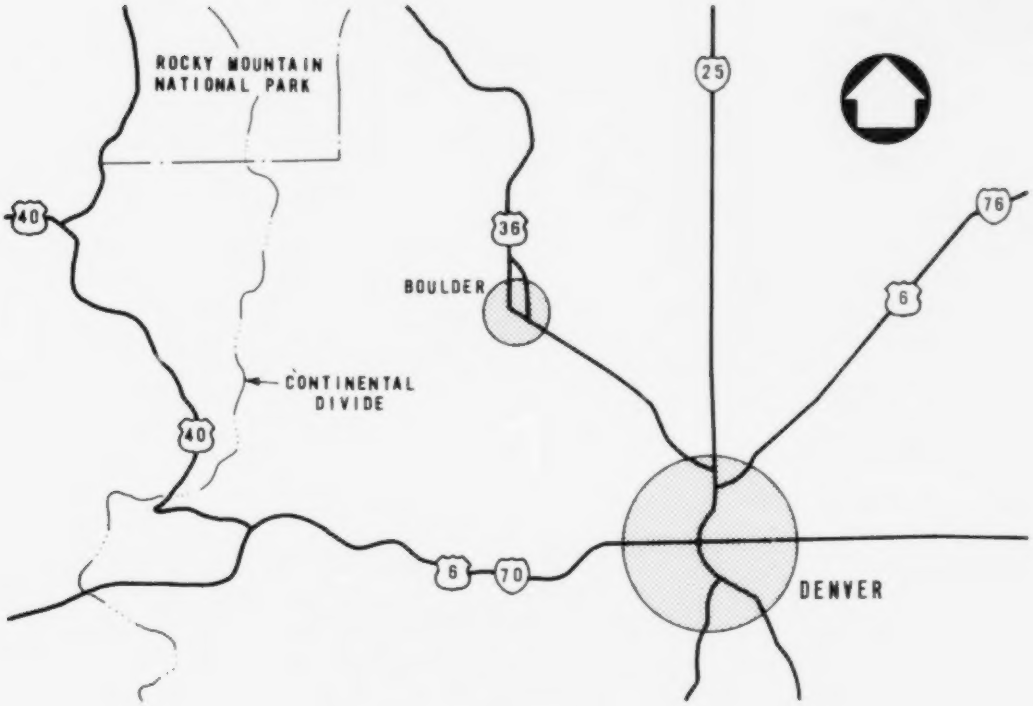


Figure 72. Boulder, Colorado.

The population of Boulder and the surrounding unincorporated communities has increased from 21,500 in 1950 to about 95,000 in 1979 [3]. This represents an increase of 442% in population over three decades, and much of the increase occurred in Boulder.

Problem Assessment

Boulder's stormwater problem is the result of increasing development on natural streams and drainageways in Boulder Creek basin. The basin is drained by the Boulder and South Boulder creeks and is nearly 950 km$^2$ (370 mi$^2$), of which about 640 km$^2$ (250 mi$^2$) is mountainous terrain. The two creeks have discharges that vary from about 0.14 m$^3$/s (5 ft$^3$/s) to over 8.5 m$^3$/s (300 ft$^3$/s) [4]. The Boulder Creek basin is shown in Figure 73.

The development density near the natural drainageways has increased significantly and has resulted in higher runoff rates. New development, especially on steeply inclined areas, has resulted in higher runoff rates causing erosion and sediment deposition problems. Portions of the developed area in Boulder are located within the 100 year flood plain and are susceptible to flooding, as shown in Figure 74.

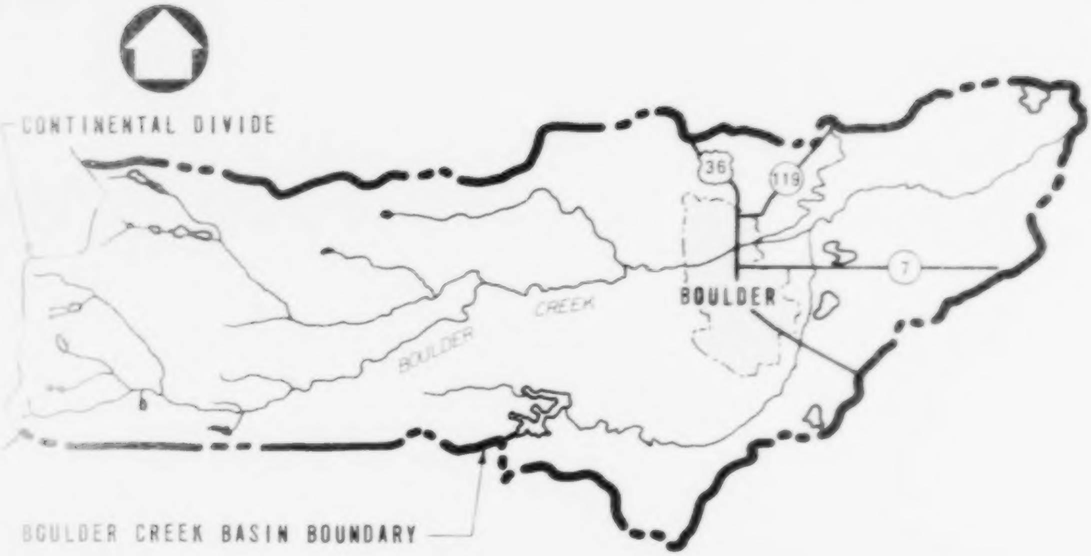


Figure 73. Boulder Creek basin.

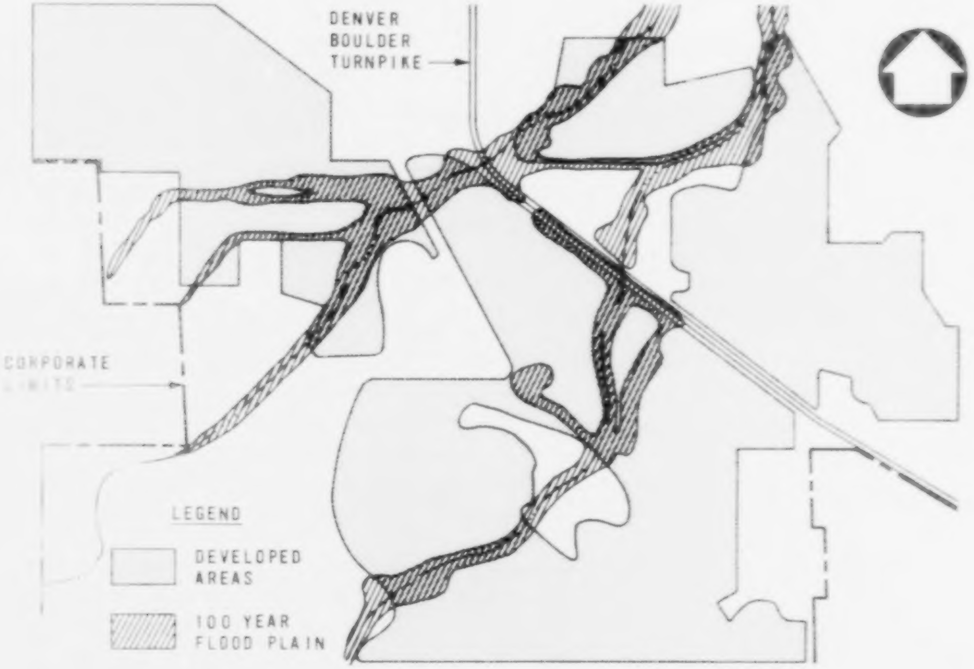


Figure 74. Potential flood hazard areas affecting a portion of the developed area in Boulder.

Limited sampling of stormwater and snowmelt runoff indicates that runoff pollutants can have a short-term influence on the streams flowing through developed areas. A sampling program conducted on a 50 ha (125 acre) drainage area with 89% residential land use was used to characterize runoff and base streamflow [1]. The runoff from the area enters the stream through a storm drain. Only one storm was sampled for rainfall runoff and up to five snowmelt runoff events were sampled to establish concentration ranges and flow weighted means. The results are summarized in Table 88.

Table 88. CHARACTERIZATION OF BASE STREAMFLOW, STORMWATER, AND SNOWMELT RUNOFF, BOULDER [1]

| | Pollutant concentration, mg/L | | | | | |
|---|---|---|---|---|---|---|
| | Base flow | | Stormwater[a] | | Snowmelt | |
| Parameter | Range | Flow-weighted mean | Range | Flow-weighted mean | Range | Flow-weighted mean |
| Total solids | 240 - 360 | 310 | 100 - 680 | 240 | 110 - 4,600 | 1,300 |
| Suspended solids | 1 - 12 | 9 | 12 - 460 | 180 | 0 - 630 | 200 |
| BOD | 0.8 - 1.4 | 1.1 | 1 - 48 | 18 | 6 - 30[a] | 10 |
| COD | 0 - 11 | 6 | 3 - 300 | 120 | 7 - 730 | 210 |
| $NO_3$-N | 4.4 - 5.8 | 4.9 | 1 - 4 | 1 | 0.1 - 6 | [illegible] |
| Total phosphorus | 0.004 - 0.04 | 0.01 | 0.02 - 0.14 | 0.12 | 0.001 - 0.35 | [illegible] |
| Oil and grease | 2 | 2 | 2 - 19 | 8 | 3 - 70[b] | [illegible] |

a. Results of one monitored storm.

b. Results of three monitored storms.

Receiving stream impacts of snowmelt runoff for one monitored event show an increase in peak concentration of 2 to 4 times for most pollutants, below the storm sewer discharge point, as shown in Table 89. Peak COD concentration showed over a 40-fold increase from above to below the storm sewer [2].

## Countermeasure Philosophy

BMPs are being implemented in Boulder to attenuate peak flows of stormwater runoff by source detention and construction or enlargement of drainageways. Ordinances were adopted to establish (1) a storm drainage and flood control utility, (2) land use planning criteria, and (3) criteria identifying the magnitude of stormwater control required.

The storm drainage and flood control utility is not only a regulatory [illegible] but also is responsible for the financing, construction, and operation and maintenance of public stormwater facilities.

Land use planning has effectively eliminated future development from areas consistently affected by flood waters and ensures the availability of drainageway easements.

Table 89. EFFECT OF PEAK SNOWMELT RUNOFF CONCENTRATIONS ON BOULDER CREEK[a] [2]

| Parameter | Peak pollutant concentrations, mg/L |  |  | Ratio of stream concentrations above and below the storm sewer outfall |
|---|---|---|---|---|
|  |  | Boulder Creek |  |  |
|  | Storm sewer | Above | Below |  |
| Total solids | 6,490 | 347 | 1,032 | 3.0 |
| Total volatile solids | 425 | 44 | 102 | 2.3 |
| Suspended solids | 1,230 | 58 | 214 | 3.7 |
| Volatile suspended solids | 330 | 25 | 84 | 3.4 |
| COD | 935 | 4 | 165 | 41.3 |
| Total phosphorus | 3.3 | 0.2 | 0.2 | 1.0 |
| TKN | 5.9 | 0.4 | 1.6 | 4.0 |
| $NO_3$-N | 4.7 | 0.2 | 0.5 | 2.5 |
| Oil and grease | 155 | 7 | 26 | 3.7 |
| Lead | 2.3 | 0.4 | 0.8 | 2.0 |
| Chlorides | 3,185 | 100 | 398 | 4.0 |

a. Values represent one snowmelt event.

Controlling stormwater and flood waters has been designated, by the city, as the responsibility of owners of property greater than 0.4 ha (1 acre). These landowners must limit runoff to the natural runoff rates existing before development and are assessed a fee based on the amount of runoff in excess of the natural runoff rate. They must also control runoff traversing their property, even though the runoff may come from other property [5].

### Implemented Controls--

The controls used to attenuate peak stormwater and snowmelt flows include improving drainageways and constructing detention facilities to reduce flooding. The construction of stormwater control facilities is only about 20% complete. Although these facilities were planned principally for flood protection, a number of the facilities also control water quality. Sediment ponds at new construction sites and stormwater detention ponds trap eroded sediment and reduce pollutants in the stormwater. A portion of the planned drainageway improvements and detention facilities to reduce peak stormflows and flooding are shown in Figure 75.

*Natural Drainageway Improvement*--Control of runoff by channelizing the drainageways to improve their hydraulics has been a popular method in this country. Boulder has widened and improved several stream channels so that the flow from a 100 year storm can pass through the city without causing excessive damage. The improvement of these drainageways complements the surrounding landscape by using a 4:1 slope on the embankment when possible, as shown in Figure 76(a).

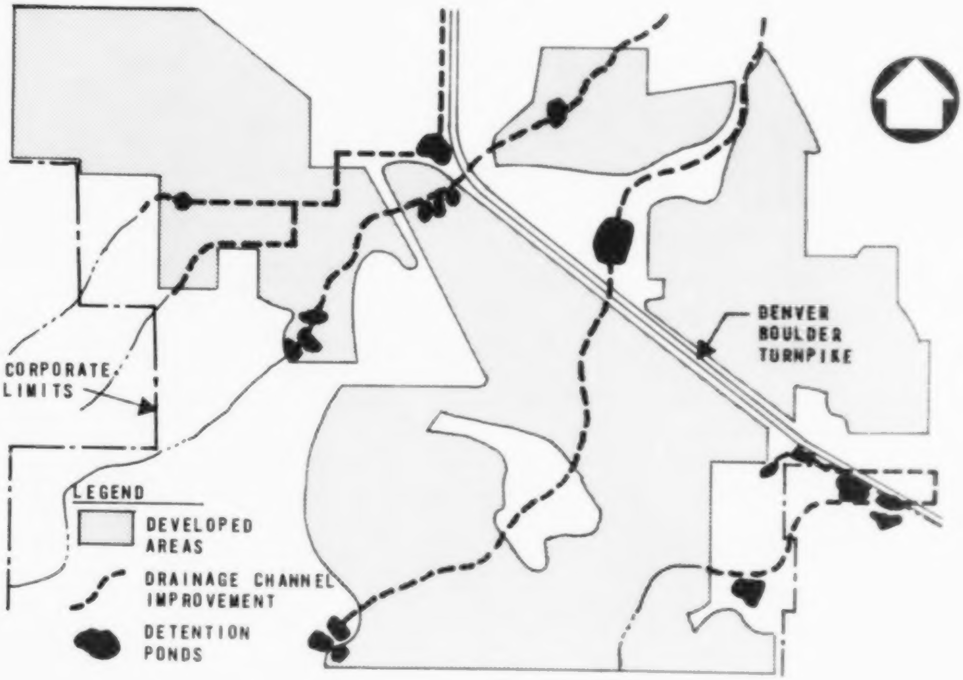


Figure 75. A portion of planned stormwater control facilities, Boulder.

Drainageway Construction--The increased development near streams has made natural drainageway improvements difficult in some portions of the city. Construction of new drainageways through these developed areas, where easements can be obtained, is an alternative to provide sufficient runoff control. When possible, these are constructed in a natural configuration for aesthetic enhancement, as shown in Figure 76(b).

Detention Ponds--Instream and offstream detention ponds are also extensively used to control stormwater. When land is unavailable for new or improved drainageways, instream and offstream detention ponds are constructed at locations upstream of confined areas.

Detention ponds provide storage and attenuate peak flows; this reduces the size requirements of the downstream drainageway. Boulder has constructed a wide variety of instream detention facilities, including detention ponds in series with restricted pipe outlets and small check dams in stream beds. Instream detention facilities are shown in Figure 77. A few developments have offstream ponds that remain partially full during nonstorm periods.

Onsite Source Detention--The most widespread method of stormwater control for private landowners is onsite source detention, using rooftop and parking lot storage methods. Each method uses the existing features of the development to store stormwater runoff with the release rate determined by the drain capacities. Examples of typical rooftop and parking lot storage are shown in Figure 78.

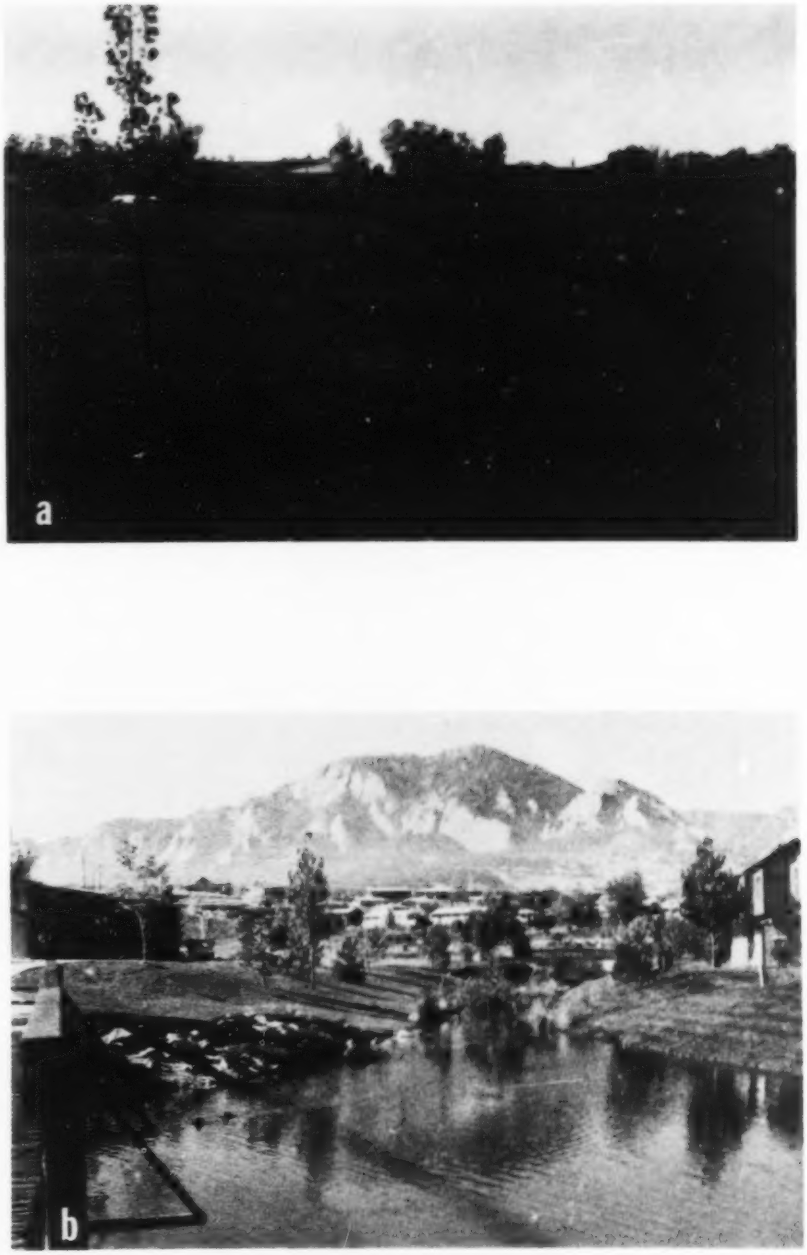


Figure 76. Drainageways in Boulder with 4:1 slopes: (a) natural drainageway improvement, and (b) newly constructed drainageway.

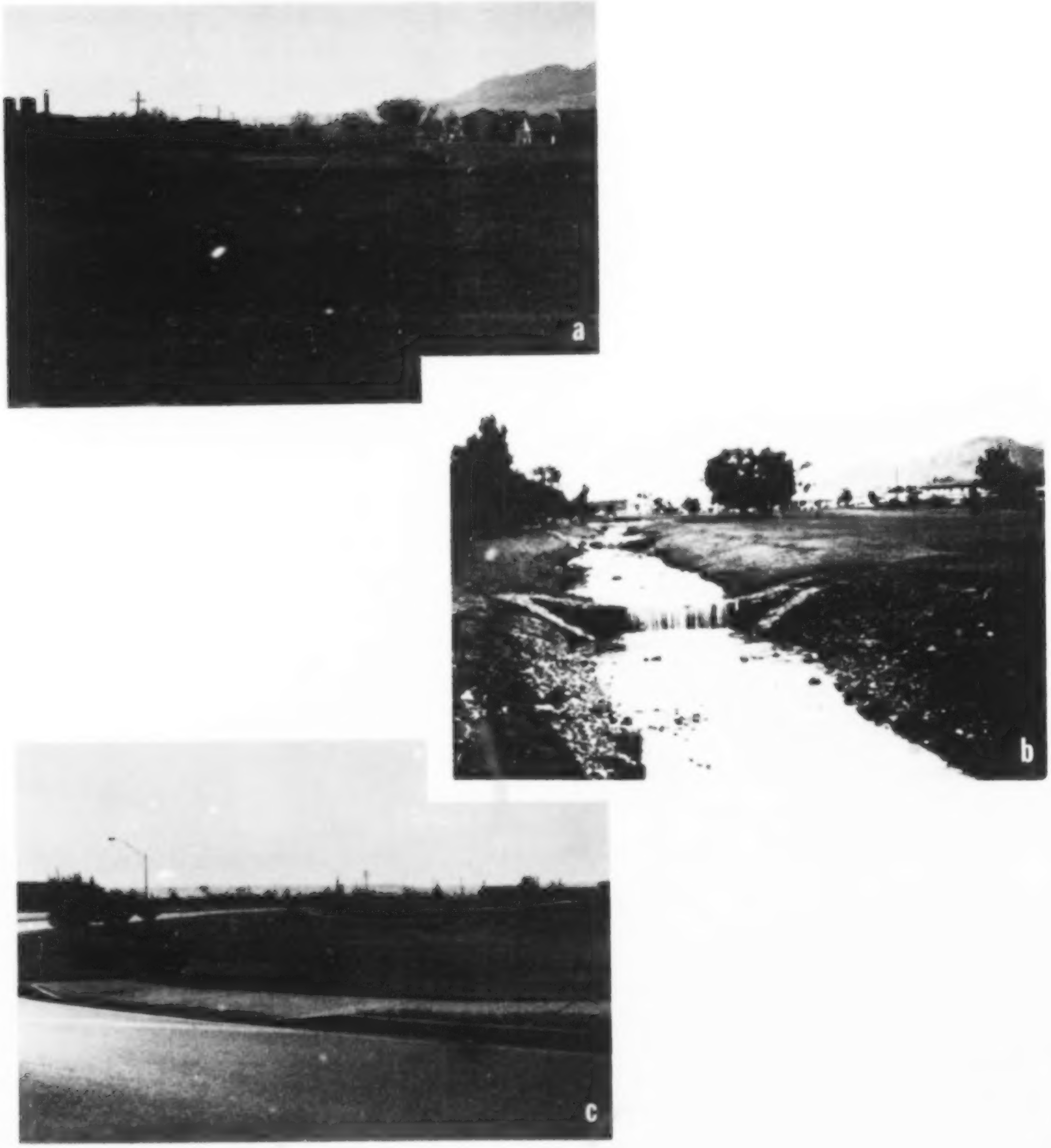


Figure 77. Stormwater detention facilities, Boulder: (a) a series of three detention ponds with restricted outlets used and maintained as pasture by local residents, (b) check dams creating storage along a natural drainageway, and (c) instream detention facility adjacent to a highway.

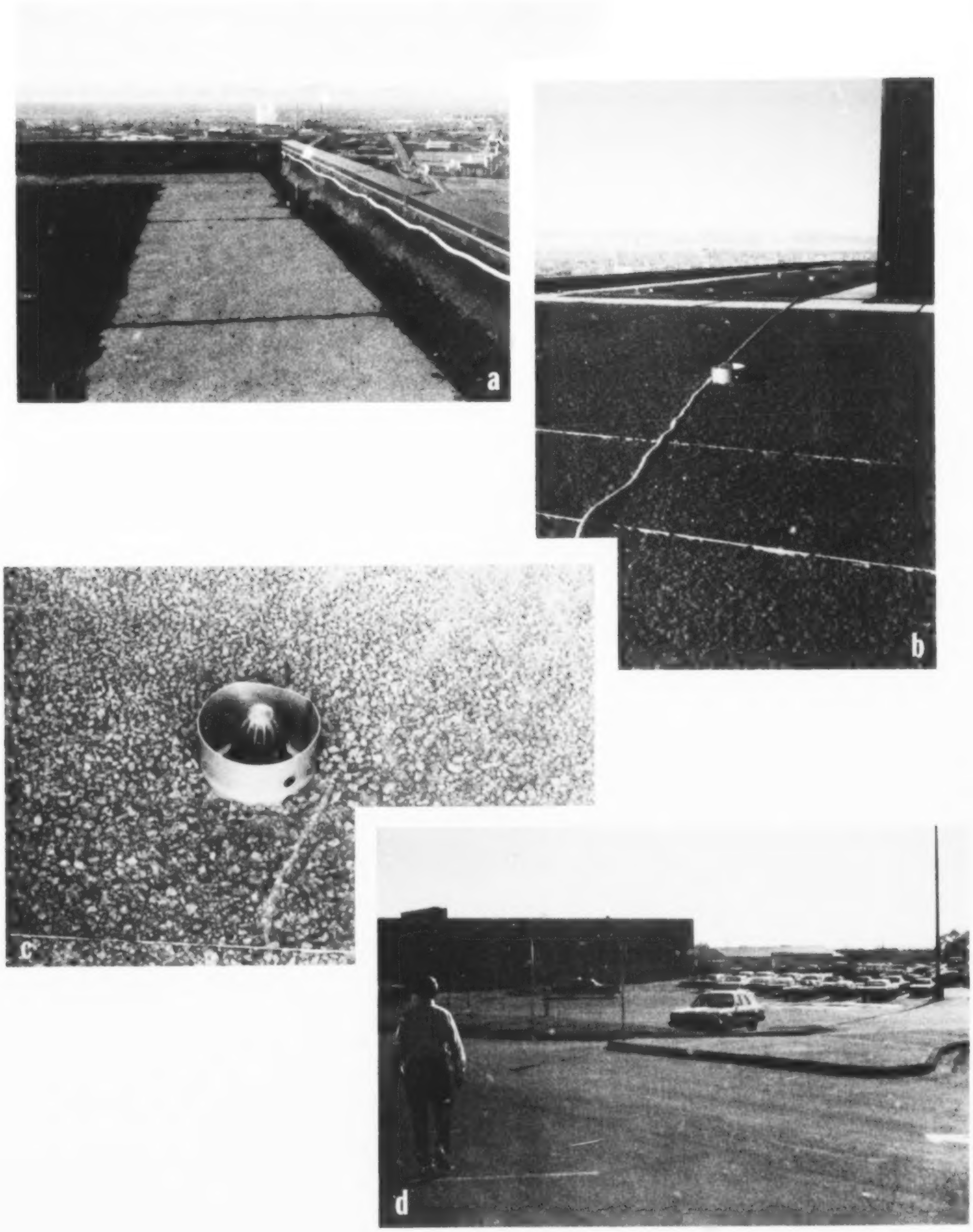


Figure 78. Onsite detention facilities: (a) rooftop storage uses potential storage volume created by parapet wall and waterproofed flashing, (b) stored volume is released through a series of drains, (c) drain has control orifice and debris trap, and (d) parking lot storage uses depression in lot to store runoff.

Sedimentation Ponds--Boulder also uses sedimentation ponds in new developments on relatively steep slopes to control erosion from road construction and home foundations. A sedimentation pond containing sediment from a new development on steep slopes with erodible soils is shown in Figure 79. Sediment is removed from the pond and used for fill and construction material as needed.

Percolation Ponds--The use of percolation ponds to control stormwater runoff is limited in Boulder. One of the percolation facilities shown in Figure 80 collects and percolates runoff from a shopping mall in a grass-covered swale partially covered with No. 2 stone. A residential area also uses a percolation pond consisting of a grass-covered depression to contain and infiltrate the stormwater.

Land Use Planning--The city has implemented land use planning to complement the facilities that control stormwater runoff. Land use planning regulates development in the 100 year flood plain and has divided it into developable and nondevelopable areas. Development is excluded from that portion of the flood plain that conveys the main portion of a 100 year flood at velocities and depths that would cause significant damage. Development, if properly protected, is allowed within the 100 year flood plain in the areas that would transport slowly moving flood waters.

Design Criteria--

The design of drainageways and detention ponds to control stormwater runoff is approached on a case-by-case basis and depends on the site conditions and stormwater problems. The stormwater problems are usually flooding and conveyance of peak storm flows. The criteria for improving or constructing drainageways are keyed to their peak flow carrying capacity rather than quality or storage/detention time criteria. The design of instream or offstream detention facilities uses flow or hydrograph routing procedures to provide adequate volume for containing peak flows at the source or instream where adequate volume is available.

Natural Drainageway Improvement--Boulder's program of stormwater runoff control emphasizes the use of natural drainageways when possible. The improvement of the drainageways to transport runoff from the 100 year flood includes the following design criteria [6]:

- Adequate capacity to contain 100 year runoff volume
- Critical velocity limited to 3 m/s (10 ft/s)
- Use of roughness factors representative of unmaintained drainageway conditions
- Control of water surface profile by construction of controlled drops or check dams
- Prevention or planning for future intrusion in the flood plain that may affect the water surface profile

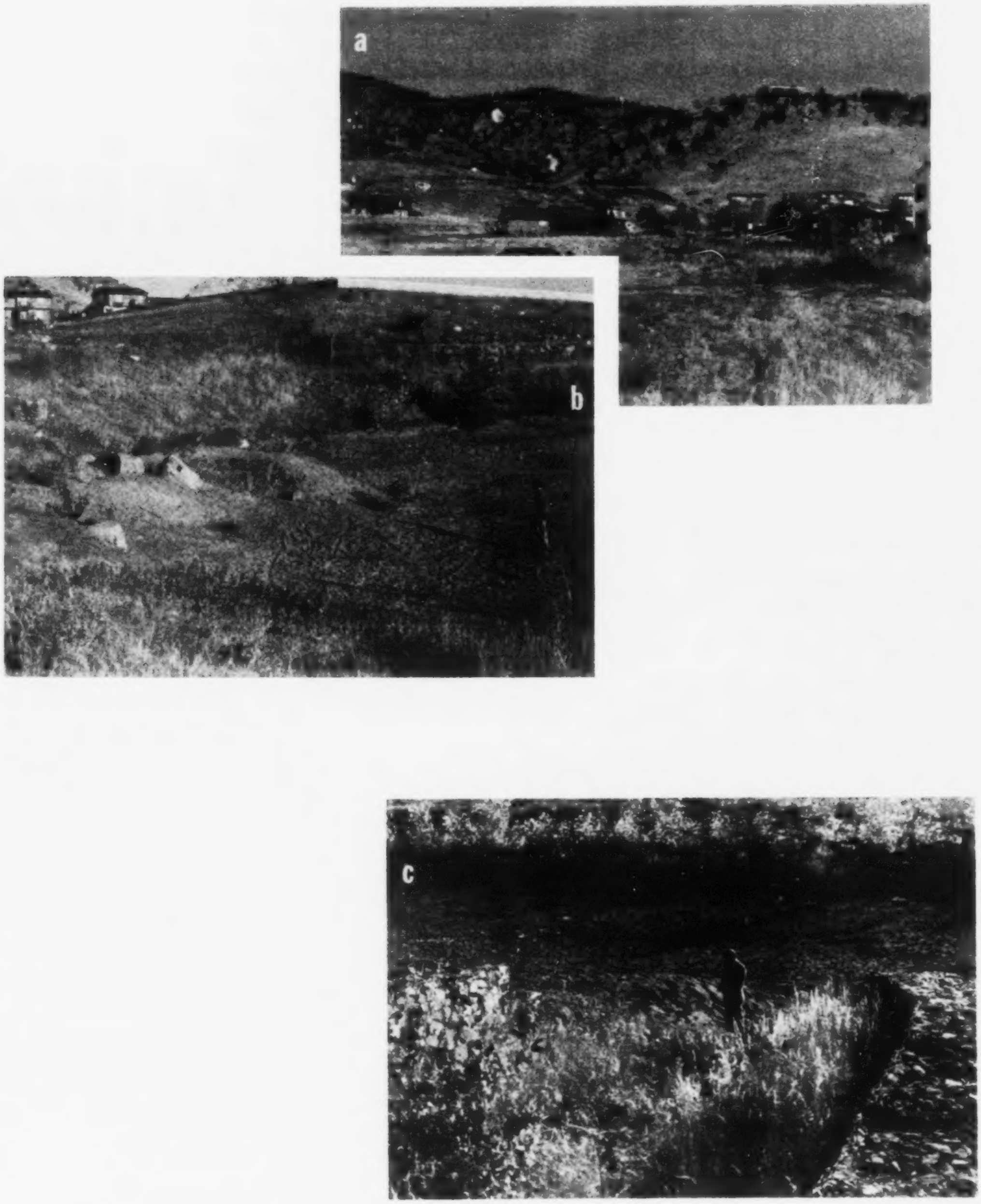


Figure 79. Sediment pond serving new residential subdivision developed on a steep erodible hillside, Boulder: (a) portion of the new development creating erosion and sediment problems, (b) sediment pond with retained sediment in the bottom, and (c) entrance channel to sediment pond protected by gabion retaining walls.

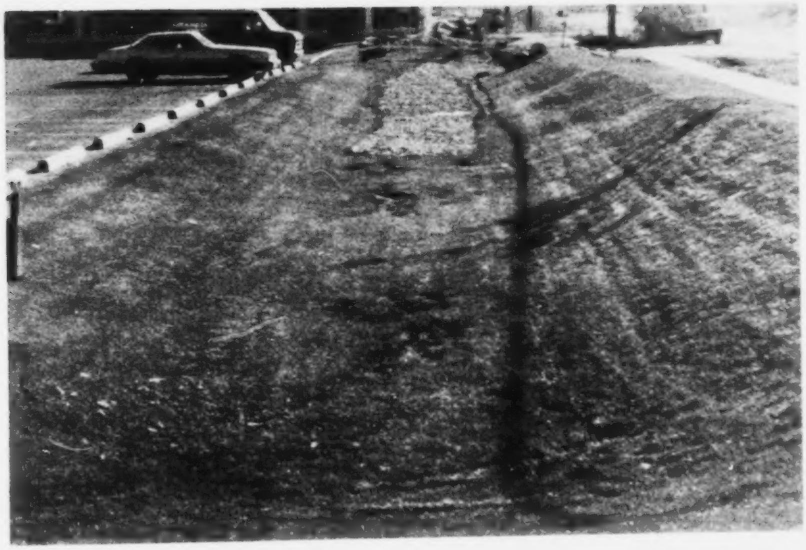


Figure 80. Percolation pond with grass-covered swale and stone-covered bottom receives runoff from a commercial area in Boulder.

Drainageway Construction--New drainageways have been necessary because of the intrusive development into the 100 year flood plain. The suggested design criteria for major grass lined channels are shown in Table 90.

Table 90. SUGGESTED DESIGN CRITERIA
FOR MAJOR GRASS-LINED DRAINAGE CHANNELS [6][a]

| Parameter | Runnoff from 100 yr storm |
|---|---|
| Velocity, V, ft/s | 2.0 ≥ V ≤ 7.5 |
| Depth, D, ft | 1.0 ≥ D ≤ 4.0 |
| Side slope | ≤3:1 |
| Horizontal curvature, ft[b] | R = 2W, R ≥ 100 |
| Freeboard, ft | 1.0 to 2.0 |
| Roughness coefficient[c] | 0.030 to 0.035 |

a. Assume that major channel will be discharging to a river or large drainage area.

b. R = radius or curvature
W = width of channel top

c. May be increased if heavy in channel vegetation is anticipated in the future.

Detention Ponds--The detention ponds are located instream and offstream, mainly in upstream areas. The design of facilities is based on the Rational Method or the Colorado Urban Hydrograph Procedure (CUHP). The Rational Method (Q = CIA), presented in numerous textbooks, is a direct relationship between rainfall and runoff for small watersheds.

The preferred design criteria follow the CUHP because of its greater dependability and detailed analysis of the storm hydrograph. The CUHP is similar to the unit hydrograph method, except that the coefficients are determined by actual field data collection and studies in Colorado.

The design of instream and offstream detention ponds has one of two goals:

- Reducing runoff rate
- Providing instantaneous storage

Detention ponds that reduce runoff rates are generally in upstream areas and use an undersized drainpipe of no less than 30.5 cm (12 in.) in diameter to regulate the flow release rate [6]. Drainpipes smaller than this tend to plug up with debris.

Detention ponds are more often used to provide instantaneous storage to attenuate the peak flow before it enters constricted areas, where flow cannot be transported within the drainageways. Design of these facilities has been limited to the size of the easement available. The release rate of these facilities is uncontrolled.

Onsite Source Detention--Boulder requires onsite detention for all developments larger than 0.4 ha (1 acre) that are not part of a larger development. The onsite detention facilities must meet standards established by the Urban Storm Drainage Criteria Manual (USDCM) [6] or general urban hydrology practice [5]. Rooftop and parking lot storage design parameters suggest a maximum stormwater storage depth of 7.6 cm (3 in.), and the rooftop drainage outlet should be large enough to release about 1.3 cm (0.5 in.) of ponded depth per hour [6].

Sedimentation and Percolation Ponds--Boulder has no specific design criteria for sedimentation or percolation ponds. Stormwater facility design usually follows suggested parameters of the USDCM.

## OPERATION AND PERFORMANCE

The operation and performance data from Boulder's program to control stormwater runoff rates are limited because the implementation of the control program is approximately 20% complete. The preliminary results indicate a reduction of flooding in the historic problem areas of the city from rainfalls of the 2 and 5 year frequency [7].

## Operation and Maintenance

The operation of stormwater facilities is divided between the city and the private sector. The city operates the facilities that transport 100 year storm flows and the instream and offstream detention facilities. The private sector is responsible for onsite detention facilities, such as rooftop or parking lot storage. Instream detention ponds may have adjustable stop logs that can regulate the water elevation of the pond.

The maintenance of stormwater runoff controls in Boulder includes practices such as:

- Debris removal in drainageways
- Landscape care
- Repair of erosion damage
- Removal of deposits in sedimentation ponds
- Rooftop storage drain maintenance
- Parking lot storage drain maintenance

The city is responsible for maintaining the drainageways, natural or man-made; the associated instream and offstream detention ponds; and the drainageway easements on private property. All other facilities on private property are maintained by the owners and usually require landscape care and debris removal.

Special attention must be taken to avoid flooding of rooftop and parking lot storage facilities because leaves or debris can plug drains. Damage to buildings may result from plugged rooftop drains. Facilities, such as those located at apartment complexes and commercial centers, also require a high degree of maintenance and landscape care to achieve an acceptable level of aesthetics. Boulder lowers its maintenance costs by allowing residents near some stormwater facilities to use the land in exchange for upkeep. An example is a detention pond where a resident maintains the facility in exchange for grazing rights for his horses.

## Performance

The Boulder control program, although only 20% completed, has reduced flooding in certain sections of the city [7]. A sedimentation pond located on Two Mile Canyon Creek has reduced the sediment loading from hillside development upstream of the facility. The large quantity of solids that must be removed from the pond annually indicates that the facility is effective. However, the annual quantity of sediment settling in the pond has not been monitored, and estimates of annual flooding are further complicated because several city departments use the sediment for construction and repair.

Although Boulder's principal goal is flow control, a dual benefit of storage is water quality control. Using the performance data on source detention and treatment efficiencies presented in previous case histories (Montgomery

County, Middlesex County, and Orange County), it is estimated that significant water quality benefits can be achieved by the facilities in Boulder. Trap efficiencies in permanent pool detention basins for most stormwater pollutants can range from 50 to 90%. Percolation facilities can reduce pollutant load to receiving waters by 100% but may contribute to groundwater pollution.

## COSTS

The construction costs of several stormwater controls were analyzed and are presented in Table 91. The cost data are limited. The costs of the sedimentation and detention ponds may not reflect the true cost since both facilities were created by the excavation of soil required as fill on other projects, and the cost for the work is not reflected in the table.

Table 91. CAPITAL COSTS OF STORMWATER CONTROL FACILITIES, BOULDER[a]

| Facility | Ownership | Storm design, yr | Area served, acre | Storage capacity, $ft^3$ | Construction cost, $ | Cost/acre served, $/acre | Storage cost, $/$ft^3$ |
|---|---|---|---|---|---|---|---|
| Sedimentation pond | Public | --[b] | 977 | 102,000 | 20,300 | 21 | 0.20 |
| Detention pond | Public | 100 | 347 | 436,000 | 90,200 | 260 | 0.21 |
| Drainageway improvement | Public | 100 | 900 | --[c] | 829,500 | 840 | --[c] |

a. ENR 3000.

b. Temporary solution, no design.

c. Drainageway improvements for 7,490 ft of channel at about $110/linear ft.

Information about the annual cost of operating and maintaining the facilities is also limited. The cost of maintaining the percolation pond that serves an area of 13 ha (32 acres) is about $250 annually.

## IMPACTS

The impacts of the Boulder program at the present stage of implementation appear minimal. Anticipated socioeconomic impacts, such as Boulder's minimal growth policy and land use planning, may be more significant than environmental impacts.

Land use controls will be the most significant results of the program. The project effectively eliminates new development in dangerous areas of the flood plain. However, this land is used as open space and provides more area for wildlife. Flood plain ordinances do allow some specialized industry, such as material-loading facilities and agriculture, to be located in the flood plain, although recreational facilities and wildlife refuges are preferred [7].

Socioeconomic impacts associated with the program are mostly positive. The stormwater controls have reduced property damage caused by floods and created more open space and recreational areas, as shown in Figure 81. Although relocation of homes and businesses within the plain is a significant problem, public acceptance of the program is high. The completion of the program is not anticipated to substantially affect demography or economic activity in the area; however, more public funding to finish the program is required.

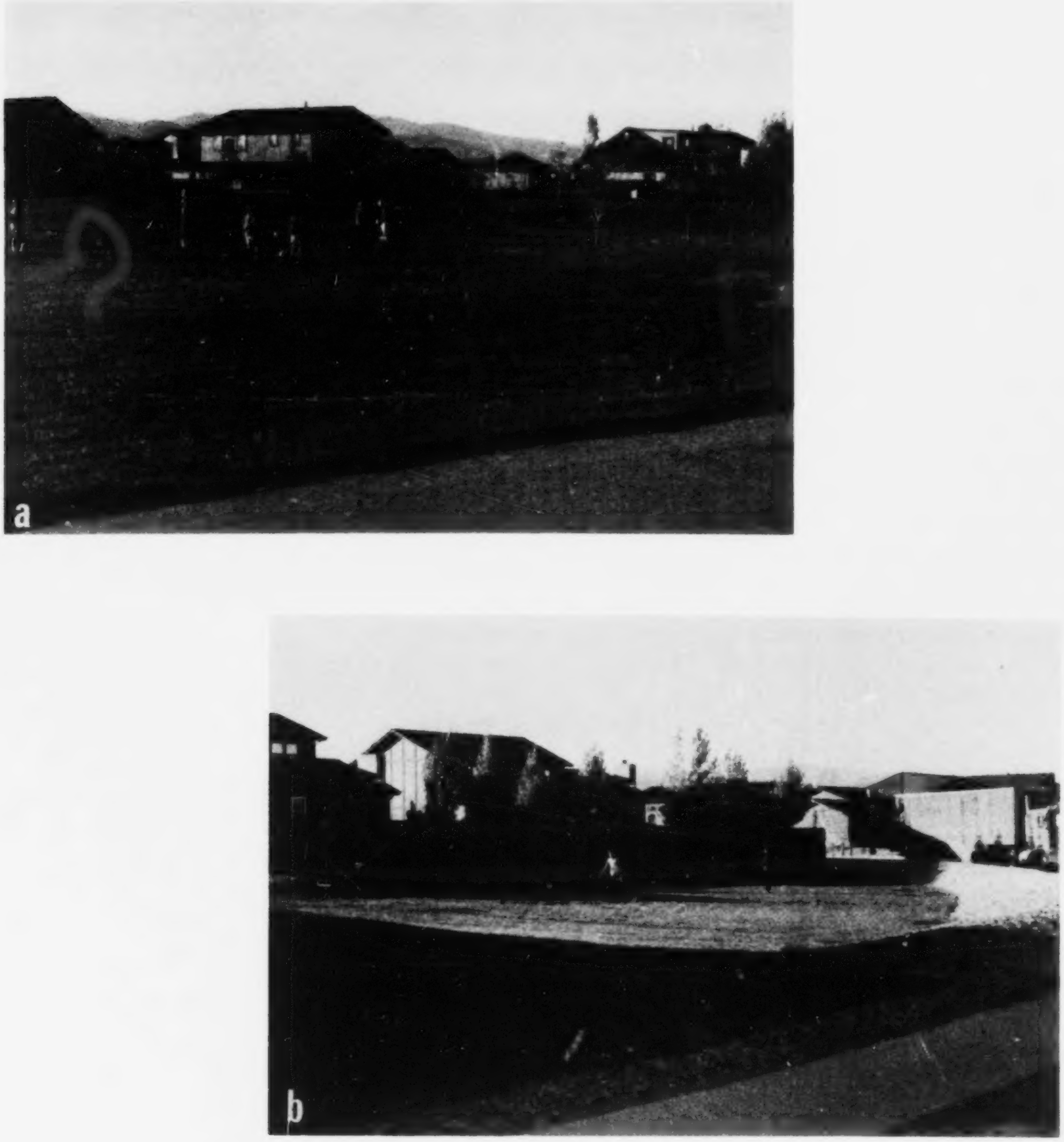


Figure 81. Multiuse stormwater facilities, Boulder: (a) park/detention pond, and (b) grassed, residential percolation pond.

# PART 2

# COMBINED SEWER OVERFLOW CONTROLS

## SECTION 12

## INLINE STORAGE CONTROL
## SEATTLE, WASHINGTON

Inline storage uses existing excess storage capacity in combined sewer interceptors and trunk sewers. Given that storage (and/or treatment) of combined sewer overflows is generally a far less costly control measure than equivalent sewer separation in large metropolitan areas, inline storage control is a particularly attractive measure where excess capacity exists and is readily adaptable to control. It has the potential for low capital investments and multiuse capabilities.

The inline storage control system in Seattle, Washington, is an example of this most promising technology. The components of the Seattle facilities include a comprehensive integrated system of collection, treatment, and disposal functions, which can be controlled by a computer. Support functions include system data collection and logging. While necessary to control flow during wet weather, these support functions also provide total system monitoring capability during dry-weather periods.

The Seattle system has demonstrated that inline storage is effective in controlling overflows within the limits of (1) the available excess storage capacity, (2) the existing system, and (3) the local characteristics. The high level of operational flexibility and reliability of system components also contributes to the effective performance of the system.

The costs of the Seattle system are competitive with the least expensive alternative offline storage/treatment systems. Also, the multiuse capabilities of the inline storage offer greater potential benefits.

Assessment of the environmental and socioeconomic impacts of the inline storage control program indicates positive improvements in the receiving water quality with minimum adverse impacts on the community. The system can also serve as the foundation for future expansion of the combined sewer program, if water quality objectives become increasingly stringent.

### CONTROL OF COMBINED SEWER OVERFLOW

The combined sewer overflow problem in Seattle is similar to that experienced in other major cities in the country with combined sewer systems. However, local characteristics are of major importance and necessarily dictate system configuration and the approach used to meet water quality goals.

## Local Characteristics

Important local characteristics to consider when selecting a method to reduce combined sewer overflows include: topography, rainfall, and land use in the combined sewered area. Seattle, a major west coast city, is bounded by freshwater lakes and marine waters consisting of bays, estuaries, and rivers of Puget Sound, as shown in Figure 82.

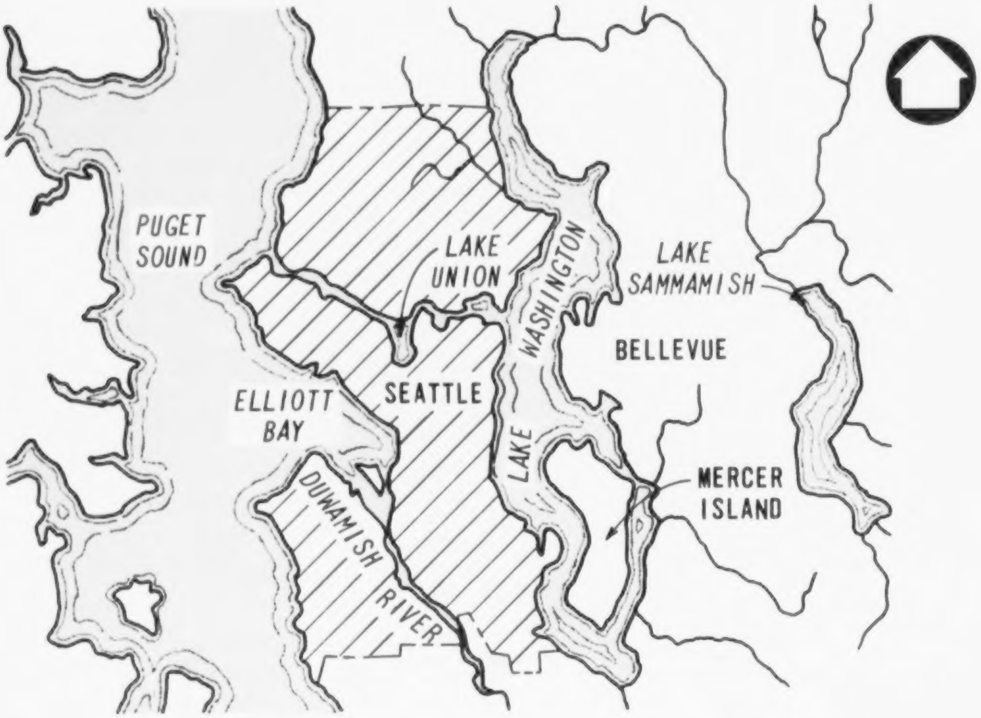


Figure 82. Seattle and surrounding receiving waters.

The Seattle topography is characterized by rolling hills of approximately 120 m (400 ft) in height. Much of the highly impervious metropolitan area of the city is located on these hills with some areas on extremely steep slopes. These characteristics are unlike those found in most cities with combined sewers in the midwest and the east.

Seattle's climate is predominately maritime, with rainfall ranging from 50 cm (20 in.) to 100 cm (40 in.) annually. Most of the rainfall occurs during the winter, October through March, and is characterized by long duration, moderate intensity regional storms. Summer storms are more localized and have shorter durations.

The service area of Seattle encompasses nearly 21,900 ha (54,000 acres), of which 6,080 ha (15,000 acres) is served by combined sewers, 9,320 ha (23,000 acres) by partially separated sewers, and 6,480 ha (16,000 acres) by sanitary sewers [1]. The total area served by combined sewers and partially separated sewers is 15,390 ha (38,000 acres).

The City of Seattle started a program of combined sewer separation in 1970 to eliminate local flooding in specific problem areas in the city [2]. The separated storm sewers either discharged directly to the receiving water or were reconnected to the combined sewer downstream of the problem area. The combined and partially separated sewer areas are shown in Figure 83.

Land use is an important factor in determining the volume of stormwater flow entering the combined sewer system. Impervious areas that have steep topography can add significant amounts of wet-weather flows to a combined sewer system. In Seattle, a large portion of the total area, 27%, is covered by streets and parking lots. Land use characteristics of Seattle are shown in Table 92.

Table 92. LAND USE CHARACTERISTICS, SEATTLE [3]

| Land use | Acres 1961 | Acres 1970 | Change 1961-1970, acres | Percent of 1970 total |
|---|---|---|---|---|
| Total land | 54,098 | 54,098 | -- | -- |
| Total developed land | 44,776 | 46,149 | 1,373 | 85.3 |
| Vacant | 7,098 | 6,184 | -914 | 11.4 |
| Water | 401 | 690 | 289 | 1.3 |
| Total residential | 19,911 | 20,307 | 397 | 37.5 |
| Single family | 17,998 | 18,161 | 163 | 33.6 |
| Multifamily | 1,913 | 2,146 | 233 | 4.0 |
| Commercial[a] | 2,137 | 2,148 | 11 | 4.0 |
| Manufacturing[b] | 3,235 | 3,666 | 431 | 6.8 |
| Public services[c] | 2,507 | 2,896 | 389 | 5.4 |
| Parks/open space | 2,703 | 2,327 | -376 | 4.3 |
| Streets, parking, and miscellaneous | 14,284 | 14,805 | 521 | 27.4 |

a. Includes retail, finance, insurance, real estates, and service categories.

b. Includes manufacturing, wholesale trade, transportation, communications, and utilities categories.

c. Includes government and education categories.

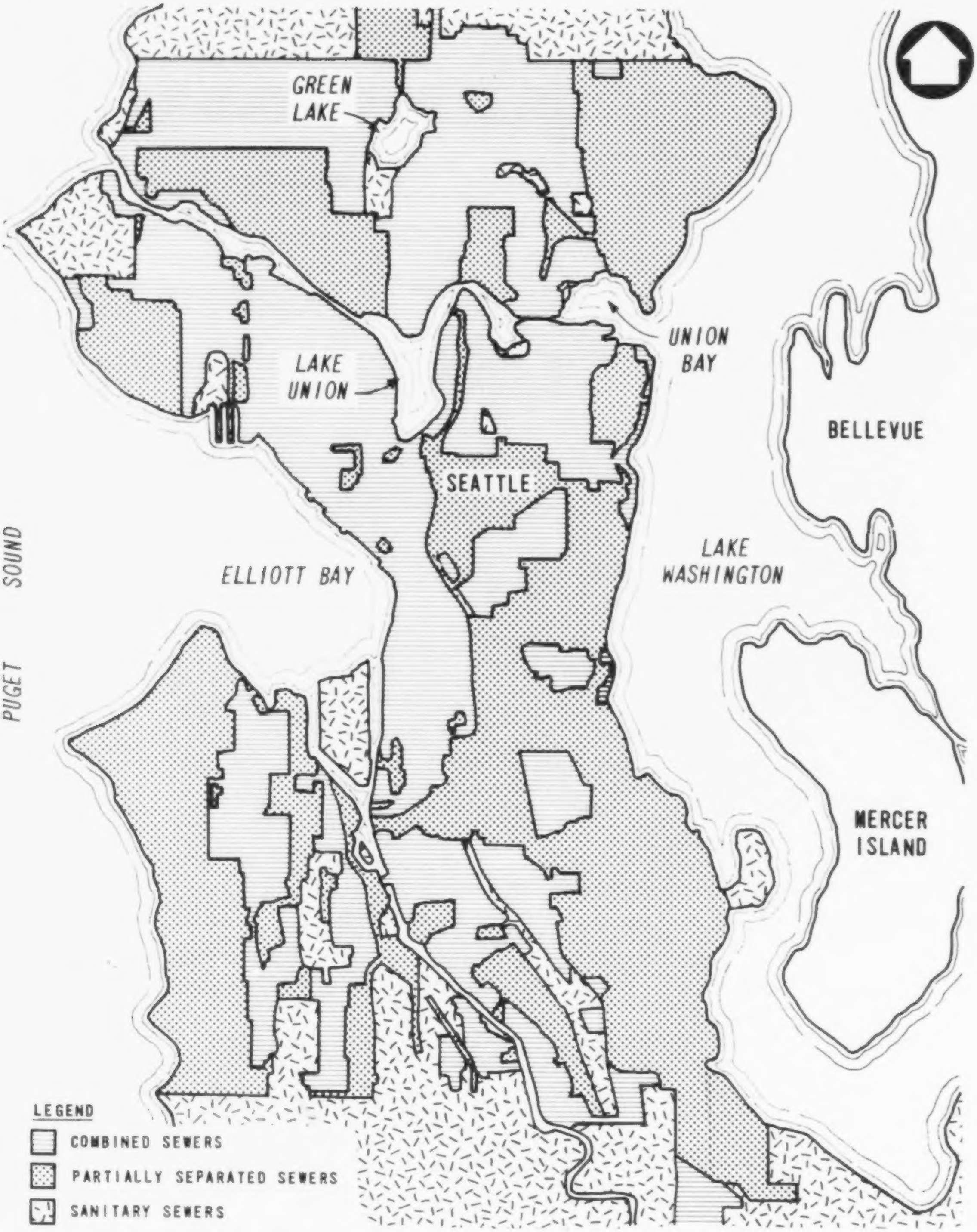


Figure 83. Combined and partially separated service areas, Seattle [4]

### Combined Sewer Overflow Problems

Combined sewer overflows, the discharge of untreated mixtures of sanitary sewage and stormwater runoff to receiving waters, decrease the quality of receiving waters and can adversely affect the environment and community, especially restricting the beneficial use of local community facilities.

#### Impacts on Receiving waters--

Adverse impacts on receiving waters are characterized by the pollutant concentration in the overflow and in the receiving water after an overflow event. A wide range of pollutant concentration values was obtained from analysis of Seattle's overflows, with the average values considered representative for the entire combined sewered area [1]. These values are shown in Table 93. The presence of these pollutants has contributed to the decline of water quality of the receiving waters surrounding Seattle.

Table 93. COMBINED SEWER OVERFLOW POLLUTANT CONCENTRATIONS, SEATTLE [1]
mg/L Except as Noted

| Parameter | Minimum | Maximum | Average |
|---|---|---|---|
| BOD | 15 | 82 | 60 |
| COD | 100 | 330 | 236 |
| Suspended solids | 141 | 296 | 217 |
| Ammonia-N | 0.5 | 1.5 | 0.9 |
| Potassium | 1.2 | 1.7 | 1.4 |
| Copper | 0.1 | 0.3 | 0.2 |
| Lead | 0.5 | 0.9 | 0.6 |
| Mercury | 0.01 | 0.01 | 0.01 |
| Chromium | 0.02 | 0.20 | 0.10 |
| Cadmium | 0.01 | 0.02 | 0.01 |
| Zinc | 0.2 | 0.5 | 0.4 |
| Total coliforms, No./100 mL | 1-8x$10^3$ | 7,000x$10^3$ | 200x$10^3$ |
| Fecal coliforms, No./100 mL | 3.6x$10^3$ | 780x$10^3$ | 250x$10^3$ |

A study is currently being conducted on Lake Washington to show the relative impacts of stormwater runoff and combined sewer overflows on benthic sediment. Results of the samples collected from the bottom sediments in areas subject to the storm influences are shown in Figure 84. Composited information includes three categories of pollutants: (1) organics (total organic carbon plus oils and grease), and total phosphorus; (2) heavy metals (copper, lead, zinc, and mercury); and (3) total chlorinated hydrocarbons (TCH) and polychlorinated biphenyls (PCBs). The length of each histogram bar, representing these categories, is relative to the lowest value found in the lake for each constituent [5]. The average strength of the various pollutants found in the sediment near the outfalls and the control areas is shown in Table 94.

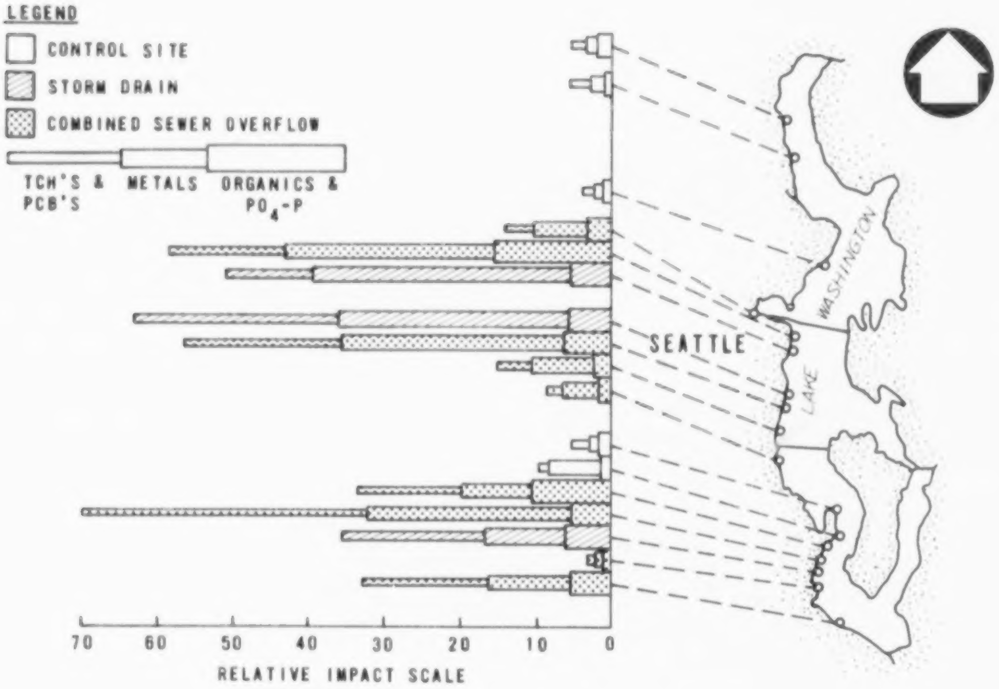


Figure 84. Comparison of the relative impacts on benthic sediments from combined sewer overflows and stormwater runoff in Lake Washington [5].

Elliott Bay and Duwamish River received most of the combined sewer overflow load from the metropolitan area of Seattle. The effects of these discharges were responsible for bacterial, floatable, sediment, and organic problems in the nearshore areas of Elliott Bay. Serious dissolved oxygen deficiencies in the Duwamish River have also been documented [6].

Impacts on Local Community Facilities--

Local problems including surcharged sewers and local flooding often developed from inadequate carrying capacity of the sewers and interceptors. In certain areas of Seattle, these problems caused basement flooding in homes and businesses resulting in health hazards and flood damage [7,8]. Adverse impacts, caused by combined sewer overflow, have also been suspected in the commercial fisheries and spawning waters in the area [1]. A summary of combined sewer overflow impacts on community and beneficial local uses is presented in Table 95.

Table 94. AVERAGE SEDIMENT POLLUTANT STRENGTHS IN LAKE WASHINGTON [5].

| Pollutant[a] | Combined sewer outfall areas | Stormdrain outfall areas | Control areas |
|---|---|---|---|
| Organics | | | |
| TOC, % | 1.2 | 1.2 | 0.4 |
| Oil and grease, mg/kg | 1,385 | 1,880 | 187 |
| Total phosphorus, mg/kg | 750 | 338 | 375 |
| Heavy metals, mg/kg | | | |
| Copper | 178 | 34 | 14 |
| Lead | 210 | 320 | 55 |
| Zinc | 250 | 140 | 75 |
| Mercury | 0.25 | 0.14 | 0.09 |
| TCH and PCBs, μg/kg | 60 | 97 | 9 |

a. All units are expressed on a dry weight basis.

Table 95. COMBINED SEWER OVERFLOW IMPACTS ON LOCAL COMMUNITY AND BENEFICIAL USES, SEATTLE [1].

| Use | Combined sewer overflow impacts |
|---|---|
| Residential | Coliforms/floatables |
| Swimming | Coliforms/floatables |
| Shellfish | Coliforms/virus |
| Fish spawning/rearing | Toxicity/suspended solids |
| Juvenile fish migration | Toxicity |
| Recreational boating | Floatables |
| Shoreline parks | Floatables |
| Commerce | Minimal |
| Industry | Negligible |

## Inline Storage Methodology

Inline storage control was implemented by the Municipality of Metropolitan Seattle (Metro) to mitigate the adverse combined sewer overflow impacts to the receiving waters surrounding Seattle. Advantages of this type of control include (1) low capital investments by using existing facilities; (2) integration with dry-weather collection, treatment, and disposal functions; and (3) adaptability of the system as the foundation for future expansion.

The inline storage also controls the existing dry-weather transport system (combined interceptors and trunk sewers) by using regulators and pumping stations to maximize the use of the excess capacity. By using and controlling this excess capacity, storm flow peaks are retained in the interceptors until downstream treatment capacity is available following a storm.

The principal goals of the inline storage program are interrelated and include the following:

- Reduction of overflow frequency
- Reduction of loading to receiving waters
- Reduction of overflow volume

The sewer separation program conducted by the City of Seattle to correct local flooding problems provides an additional benefit to the inline storage system. The separation project has reduced annual overflow volumes by an estimated 1.14 million $m^3$ (0.3 billion gal) [2], thus reducing the flow to be controlled by the inline storage system.

Facilities Description--

The inline storage control system was developed from a portion of the regional interceptor collection system and serves areas primarily affected by combined sewers. The inline storage system uses 19 regulator stations and four pumping stations to provide an estimated 86,260 $m^3$ (22.79 Mgal) of storage [3,9].

Regulators and pumping stations, key components of the inline facilities, control the unused storage capacity. Regulators are usually located at the intersection of interceptor and trunk sewers; they provide storage in the trunk sewers and control flows from the trunk sewer either into the interceptor or to a receiving water if the interceptor is full. A summary of the system storage capacities upstream from regulator and pumping stations is shown in Table 96.

Seattle's regulator stations are mechanical systems with motor or hydraulically operated gates and can be controlled statically or dynamically or by centralized remote dynamic control. In the local static control mode, the regulator gates are set at a single preset elevation to control flows. Local dynamic control, the multipositioning of the regulator gates by an operator, achieves the desired flow control in response to varying flow conditions as monitored by the central control facility. Remote dynamic control of the regulators is achieved by supervisory or automatic program control from the centralized computer facility.

Pumping stations are also located along the interceptor system to assist in transporting wastewater flows. These stations, in addition to the regulators, provide control of the inline system and are controlled from the central computer facilities. Regulator and pumping station facilities are shown in Figure 85.

System Monitoring--System monitoring is required for remote dynamic control of pumping stations and regulators. The regulator and pumping stations each have over 20 monitoring points to supply information for control logic and equipment operation. In addition, water quality data and data from a network of 11 rainfall gages are collected and used as input to the control function.

Table 96. INLINE STORAGE POTENTIAL, SEATTLE [3,9]

| Type of storage | Maximum achievable storage, Mgal |
|---|---|
| Trunk storage | |
| Hanford No. 1 | 0 |
| Hanford No. 2 | 3.50 |
| Denny Way | 0.60 |
| West Michigan | 0.01 |
| Chelan | 0.40 |
| Eighth Avenue | 0.30 |
| Harbor | 0.02 |
| Lake City | 2.60 |
| Lander | 0.80 |
| Connecticut | 0.50 |
| Dexter | 0.30 |
| King | 0.05 |
| North Michigan | 0.60 |
| Norfolk | 0.40 |
| Brandon | 0.30 |
| University | 1.90 |
| Montlake | 0.70 |
| Ballard | 0.60 |
| Interceptor storage upstream from pumping stations | |
| East Marginal | 0.10 |
| Duwamish | 2.00 |
| Interbay | 5.80 |
| Matthews Park | 1.40 |
| Total | 22.79 |

Computer Control--Seattle's computer-controlled inline storage is currently the most sophisticated system of inline storage. The computer system, CATAD (Computer Augmented Treatment and Disposal), is a reactive model, rather than a predictive model, and uses the monitoring information received to operate the system. CATAD monitors pumping stations, regulator stations, sewage treatment plants, and rainfall gages and uses programmed information such as storage capacity of sewers, related overflow points (with the least damaging effects), and treatment plant flow capacity to provide the control operation. The central computer console can monitor up to seven regulators, pumping stations, and rain gages simultaneously. CATAD's central terminal facilities include a computer, supporting hardware, control console, interceptor and data display map, data loggers, and event recorders.

CATAD also has two satellite control terminals, one at the West Point treatment plant and one at the Renton treatment plant. These terminals are a

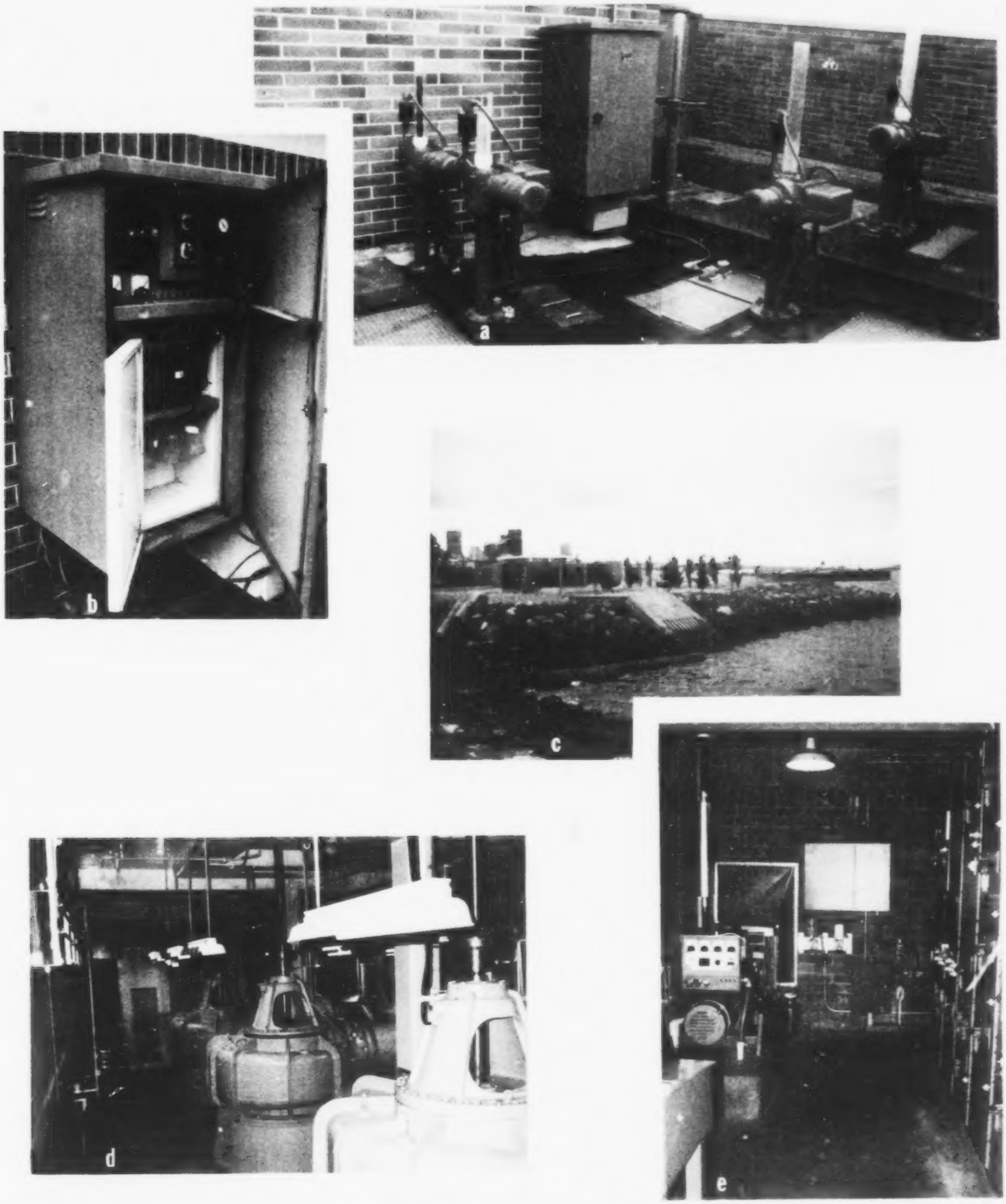


Figure 85. Regulator and pumping station facilities, Seattle: (a) exterior of regulator station showing motorized regulator gates, (b) typical and automatic sampler at regulator stations, (c) Denny Way regulator outfall, (d) typical pumping station, and (e) interior of regulator station.

smaller version of the central control console and can monitor and control only one regulator or pumping station at a time. The terminal at the West Point treatment plant uses the CATAD computer; the terminal at the Renton treatment plant operates from its own minicomputer. Seattle's computer facilities are shown in Figure 86.

### Implementation and Design Considerations

The storage potential of the interceptor and trunk sewer is the basis for implementing an inline storage program. With limits on the amount of available storage in the system, only a given level of control is possible. If higher levels of overflow control are needed, several options are available for increasing the use of the storage capacity or providing additional storage.

Increasing the density and type of system monitors, such as rain gages and level sensors coupled with sensitivity analyses of the response of various segments of the inline system, can be used to maximize the existing capacity. Increases in control sophistication achieve similar results: local control to remote or automatic control, or reactive control to predictive control.

Using the existing inline storage as the basic system, implementation considerations for additional storage could include:

- Complete or partial sewer separation
- New inline storage facilities
- Offline storage
- Source storage or BMPs

## OPERATION AND PERFORMANCE

The operation and performance of Seattle's inline storage system have provided a level of control consistent with planned goals to reduce the impacts of combined sewer overflows. Systemwide operation has evolved through increasingly sophisticated operational modes to its present mode of combined supervisory/automatic computer control. Optimization of the system's performance is keyed to knowledge and experience of the system's characteristics and response to rainfall events.

### System Operation

The present mode of operation (supervisory/automatic computer control) has evolved from local static control of the regulators and pumping stations to fully automatic centralized computer control. Under supervisory/automatic control, the CATAD system is operated continuously by supervisory personnel during the normal 40-hour workweek and is returned to automatic computer control the rest of the time, irrespective of storm conditions.

The system routes, optimally stores, or permits overflows, recognizing each segment or portion of the inline system responds differently to rainfall (different slopes, capacities, and configurations). Depending on the size and type of storm, or frequency of storm recurrents, the system can create

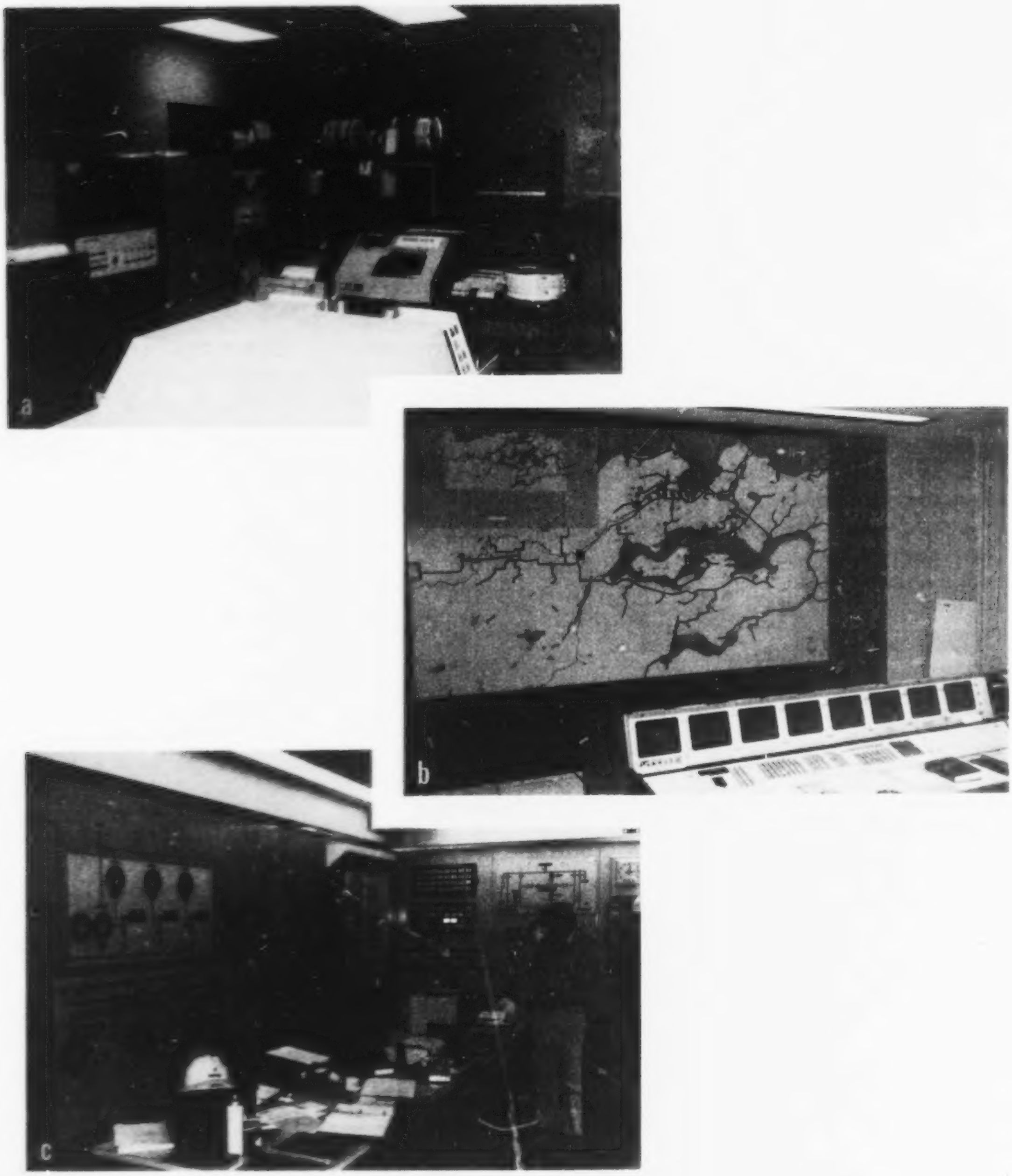


Figure 86. Seattle's computer facilities. (a) CATAD computer hardware, (b) CATAD central control console and display map, (c) CATAD satellite control terminal at West Point sewage treatment plant.

storage capacity in certain critical areas to relieve overburdened portions of the system when full.

Preferential use of outfalls is also possible when overflows occur during large storms. A priority system of allowing overflows in selected locations is based on potential impacts and capacity of the receiving water to handle the discharge (such as tidal flushing capacity).

The system can respond to various levels of rainfall. Small storms that can be totally stored in the system are retained and pumped to the West Point treatment plant as treatment capacity is available. For larger storms, only a portion of the volume is retained. For these storms, first flush materials containing heavy pollutant loads may be diverted to the interceptor for treatment. Storms of long duration may be totally stored or have a large portion stored, depending on downstream treatment capacity and frequency of rainfall. System operation and performance, however, show reduced effectiveness when large areawide storms hit the system when full from a previous storm.

System operation during dry-weather periods provides valuable surveillance and monitoring information in addition to the dry-weather flow control capability.

## System Performance

The inline storage system performs better on storms with small rainfall volumes and is less effective on storms with large rainfall volumes that occur less frequently. Both the inline storage system and the sewer separation program have reduced combined sewer overflow volumes and frequency.

### Overflow Volume and Loading Reduction--

The annual volume of combined sewage retained in the system and sent to treatment and stormwater removed from the system through the separation project has totaled 3,400,000 $m^3$ (0.9 billion gal). However, the pollutant loads contained in the stormwater from the separate sewers are still discharged to the receiving waters. Inline storage was responsible for an estimated reduction of 2,270,000 $m^3$ (0.6 billion gal), or 32% of the total annual precontrol overflow volume, as shown in Figure 87 [2]. Implementation of additional storage or controls to integrate with the inline system are planned for further reduction of the overflow volume.

Using the annual volume reduction (32% for inline storage) together with the average combined sewer overflow pollutant characteristics (Table 93), an estimated 493,000 kg (1,086,000 lb) of suspended solids and 136,000 kg (300,000 lb) of BOD were prevented from entering the receiving waters and were sent to treatment. Estimated average loading decreases of 58% ammonia and up to 76% COD have previously been reported for the inline storage system [10].

By April 1967, program implementation had reduced the amount of raw sewage entering Lake Washington by 98% [6]. Removal of all sewage nutrients from the receiving waters surrounding Seattle is not possible because of the limited storage capacity of the interceptors and trunk sewers. The largest nutrient and bacterial loadings occur during the winter months. Winter storms may result in more than 50% of the sanitary flows being discharged at overflow points before reaching the West Point treatment plant [11].

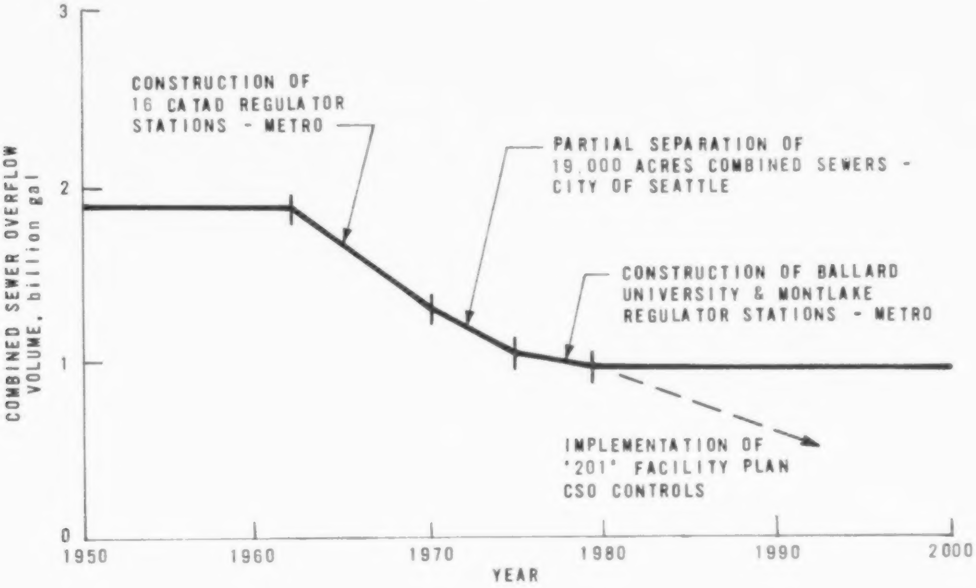


Figure 87. Reduction of combined sewer overflow volume, Seattle [2].

Overflow Frequency Reduction--

The frequency of overflows has been significantly reduced to approximately 40 overflows per year [3,12]. The occurrence of overflows during the summer recreation period have been limited to five or six. The monitoring of overflows did not begin until 1970 with the installation of CATAD components at regulator stations. Before 1970, the exact number of overflows was unknown.

Impact of Operational Mode on System Performance--

The mode of operation has a significant impact on the efficiency and performance of inline storage. The system performance was analyzed for three modes of operation: local dynamic, supervisory, and combined supervisory/automatic computer control. A performance comparison, using the total storm rainfall and the total system overflow volume per event from records collected from the CATAD monitoring system for the period 1970 to 1976, is shown in Figure 88. Over 5,000 individual data points were analyzed to develop the performance relationships. The supervisory performance regression line indicates comparable efficiency to that of supervisory/automatic computer control. This is attributed to operator knowledge, experience, and ability to predict system response, providing a high level of control approximately matching that of the control provided by the conservative model used in the automatic mode.

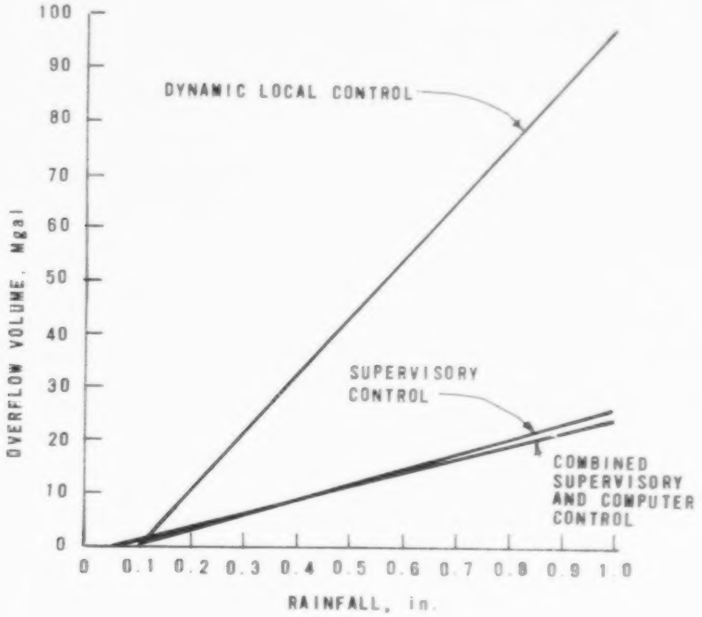


Figure 88. Comparison of inline system efficiency under three modes of operation, Seattle.

## Operation and Maintenance

### System Reliability--

Seattle's inline storage system has demonstrated a high degree of reliability in system operation with computer downtime estimated at 1 to 2%. The mechanical system reliability exhibits the commitment to preventive maintenance to ensure that all system components are in operation condition.

Regulators, treatment facilities, and pumping stations are monitored and have fail-safe controls installed. These safeguards include duplication of data sent to the computer to ensure correct information transfer and automatic monitoring of several integral parts of each station to detect an emergency situation or an incompatible response for the operating mode. When computer control is lost at regulator stations, the station automatically reverts to local control rather than cease operation altogether. Similar backup systems are available for loss of power to a station and include auxiliary engine-generator sets and manual overrides in the motor-operated gates.

System Maintenance--

System maintenance is usually one of the first tasks cut back when there is a lack of time or budgeted funds; especially when the facilities are used only to control wet-weather flows a part of the time.

The Seattle system, however, is a relatively low maintenance facility because inline storage uses existing sewer facilities. The portions of the system that require regular maintenance include the regulator and pumping stations and the associated computer hardware. Because of the complexity of the computer maintenance required, Metro contracted with a private company to maintain the computer but the supporting facilities and telemetry appurtenances are maintained by Metro personnel. Metro has developed specialized teams of maintenance technicians trained in the fields of instrumentation, engine maintenance and repair, electricity, mechanics, and general maintenance. Maintenance responsibilities are divided into two nonoverlapping main categories of (1) telemetry control units and equipment contained within the unit, and (2) all other station equipment.

Operation and Maintenance Problems--

The preventive maintenance program and CATAD's monitoring system have limited major operation and maintenance problems by identifying and isolating defective equipment or control losses. Metro's specialized technicians usually solved the operation and maintenance problems with the exception of telemetry loss due to telephone circuit difficulties or computer-related problems [9].

Other problems specifically relating to design or installation of the system that can reduce the performance of the facilities or cause a loss of control include:

- Lack of accuracy of, or improper control ranges of, instrumentation equipment
- Improper installation or calibration of sensor or mechanical hardware
- Corrosion problems with metal instruments and mechanical devices
- Odor problems resulting from trapped stagnant water in the system, such as the conduits from the regulators to the overflow structures

The West Point treatment plant experiences solids handling problems during the rainy season. During storms, the plant can receive up to a 100% increase in suspended solids load over the average daily rainy season load of about 90,800 kg/d (200,000 lb/d). The estimated increase in load resulting from the captured storm flows in the inline system is between 9,100 to 13,000 kg/d (20,000 to 30,000 lb/d). These increased loads can stress the efficiencies of the plant sedimentation tanks and can increase maintenance problems, such as grit removal. Grit removal averages 4,090 kg/d (9,000 lb/d), but during storms it can increase by up to 4 times resulting in an additional 5 $m^3/d$ (7 $yd^3/d$) of grit that must be handled and disposed of.

## ECONOMIC AND ENVIRONMENTAL IMPACTS

Inline storage offers a most promising, cost-effective alternative for combined sewer overflow control. Capital costs of inline storage control are low, and intangible cost benefits of the system from multiuse capabilities, although not as easily identified, are also a positive consideration. Dwindling resources, increasing costs, and greater public awareness of environmental issues require facilities to be cost effective and have minimum adverse impacts on the community and the environment.

### Costs of Inline Storage

System implementation costs include capital and annual operation and maintenance. Modification of the existing system to provide areawide control also includes combined sewer separation work by the City of Seattle, affecting approximately 43% of the Seattle metropolitan area. Major resource use is limited to energy consumption to operate the system and to those resources used to construct the system.

#### Capital Costs--

Capital costs of the inline system included: (1) modification or construction of the regulator and pumping stations; and (2) installation of the computer and peripheral control equipment, telemetry and control interfacing, and systemwide surveillance instrumentation.

Modification and construction costs of Seattle's regulator stations ranged from approximately \$200,000 to \$1,200,000 [9]. A summary of the regulator costs, including engineering, is presented in Table 97.

Total capital costs for the inline storage system and the sewer separation project are presented in Table 98. Inline system capital costs are broken down into costs for regulators, pumping stations, and computer facilities (including software). Of the \$924,000 (ENR 3000) engineering costs for the computer system, approximately 20% represents computer program development [9]. Costs of installing only monitoring instrumentation and associated telemetry for pumping stations are estimated at \$14,000 to \$19,000 per station.

#### Operation and Maintenance Costs--

Operation and maintenance costs of Seattle's computer-controlled inline storage system were estimated to be approximately \$440,000 per year, and include parts, material, labor, and utilities. A major portion of this cost is utility expenses. Typical utility charges for electricity have approached 50% of the annual operation and maintenance cost for pumping stations. An estimate of the operation and maintenance costs for the remote control stations and the central computer facility, adjusted to ENR 3000, is presented in Table 99 [9].

Table 97. SUMMARY OF REGULATOR STATION MODIFICATION AND CONSTRUCTION COSTS, SEATTLE[a]

| Regulator station | Construction or modification costs[b] | Engineering costs | Total costs |
|---|---|---|---|
| Hanford No. 1 | $ 276,500 | $ 41,400 | $ 317,900 |
| Hanford No. 2 | 701,700 | 109,100 | 810,800 |
| Denny Way[c] | 1,132,400 | 128,000 | 1,260,400 |
| West Michigan | 161,700 | 24,200 | 185,900 |
| Chelan | 196,700 | 29,500 | 226,200 |
| Eighth Avenue | 177,600 | 26,600 | 204,200 |
| Harbor | 210,300 | 31,600 | 241,900 |
| Lake City | 324,200 | 94,400 | 418,600 |
| Lander | 623,900 | 95,500 | 719,400 |
| Connecticut | 390,800 | 60,400 | 451,200 |
| Dexter | 928,600 | 139,300 | 1,067,900 |
| King | 233,200 | 37,000 | 270,200 |
| North Michigan | 256,900 | 38,500 | 295,400 |
| Norfolk | 240,000 | 36,000 | 276,000 |
| Brandon | 207,800 | 31,000 | 238,000 |
| University | 1,036,800 | 155,500 | 1,192,300 |
| Montlake | 709,600 | 106,400 | 816,000 |
| Ballard | 669,600 | 100,400 | 770,000 |
| Total | $8,478,300 | $1,284,800 | $9,762,300 |

a. ENR 3000.

b. Including taxes.

c. Includes two regulators in one station.

Table 98. TOTAL SYSTEM CAPITAL COSTS FOR INLINE STORAGE AND SEWER SEPARATION, SEATTLE[a]

| System | Average cost per station | Total cost |
|---|---|---|
| Inline storage | | |
| Regulator stations | $513,800[b] | $ 9,762,000 |
| Pumping stations modifications | 68,500[c] | 274,000 |
| Computer facilities and interfacing | -- | 5,717,000 |
| Total | -- | $ 15,753,000 |
| Sewer separation project | -- | $147,810,000 |

a. ENR 3000.

b. Average for 19 regulators.

c. Average for 4 pumping stations.

## Table 99. ESTIMATE OF INLINE STORAGE OPERATION AND MAINTENANCE COSTS, SEATTLE[a] [9]

| | Labor and fringes | Utilities | Other | Total |
|---|---|---|---|---|
| Remote stations | | | | |
| Operation | $143,900 | $101,600 | $ 2,300 | $247,700 |
| Maintenance | 58,700 | -- | 24,300 | 83,100 |
| Subtotal | $202,700 | $101,600 | $26,500 | $330,800 |
| Central computer facilities | | | | |
| Operation | $ 60,600 | $ 6,300 | $12,000[b] | $ 78,900 |
| Maintenance | 31,400 | -- | -- | 31,400 |
| Subtotal | $ 92,000 | $ 6,300 | $12,000 | $110,000 |
| Total | $294,700 | $107,900 | $38,500 | $441,100 |

a. ENR 3000.

b. Approximately 75% of this cost is building rental.

Operation and maintenance costs for the first 5 months of 1978 were used to estimate annual costs for various sizes of regulators, pumping stations, and CATAD telemetry control components, as shown in Table 100. The sizes of these units are based on dry-weather capacities.

## Table 100. ESTIMATES OF ANNUAL OPERATION AND MAINTENANCE COSTS OF INLINE STORAGE FACILITIES BASED ON ACTUAL 1978 FIGURES, SEATTLE[a]

| System component | Capacity, Mgal/d[b] | Projected annual operation and maintenance cost, $ |
|---|---|---|
| Regulators | 2-8 | 3,100-4,000 |
| Pumping stations | | |
| Small | 12-15 | 8,200-13,400 |
| Large | 45-50 | 100,000 |
| Computer systems[c] | | |
| CATAD telemetry control units and modules | | 18,000 |
| Telemetry and circuits | | 12,000 |

a. ENR 3000.

b. Dry-weather flow capacity.

c. Contracted maintenance for 42 stations.

Since these facilities also function during dry-weather periods, operation and maintenance costs may be shared or apportioned between the dry-weather function and the wet-weather function.

## Cost Effectiveness of Inline Storage

A comparison of the cost effectiveness of Seattle's inline storage and sewer separation project is presented in Table 101. For equivalent results of combined sewer overflow volume reduction, inline storage is over 18 times more cost effective than sewer separation. On a per acre basis, sewer separation is over 6 times as expensive as inline storage. However, both the inline system and the separation project must be viewed as a combined areawide effort that was made to solve two different problems: overflow control and local flooding. Each countermeasure was unique to each problem and both resulted in overflow reduction--inline storage would have little effect on reducing local flooding.

Table 101. COST EFFECTIVENESS OF COMBINED SEWER OVERFLOW COUNTERMEASURES, SEATTLE[a]

| Control measure | Total capital costs, $ | Contributing area, acres | Annual combined overflow volume reduction, Mgal | Excess storage capacity, Mgal | Cost, $/acre | Overflow volume reduction cost, $/Mgal | Storage cost, $/gal |
|---|---|---|---|---|---|---|---|
| Inline storage | 15,753,000 | 15,000 | 600 | 22.79 | 1,050 | 26,260 | 0.69 |
| Sewer separation | 147,810,000 | 23,000 | 300[b] | -- | 6,430 | 492,700 | -- |
| Combined inline and separation projects | 163,563,000 | 38,000 | 900 | -- | 4,300 | 181,740 | -- |

a. ENR 3000.

b. Still allows stormwater volumes and pollutants to be discharged.

Unit costs of inline storage, on an area basis, can range from Seattle's system costs of $2,590/ha ($1,050/acre) for a highly sophisticated system, down to as low as $170/ha ($70/acre) for simple, locally controlled dam devices [10]. The characteristics and capacity of the existing system would have significant impacts on the unit costs of inline storage.

## Multiuse Benefits

Seattle's CATAD system has many multiuse benefits that have not been assigned a dollar value; these include systemwide monitoring and surveillance, dry-weather flow control, and system maintenance.

Seattle's CATAD system is operated continuously during both dry- and wet-weather periods. Dry-weather operation of the system has provided Metro a total system monitoring capability, and combined with surveillance of the system facilities, an awareness of total system operation. System monitoring also provides instant monitoring of equipment failure. The immediate identification of an equipment failure allows repair crews to correct the problem before a station is severely damaged. The ability to watch and control the dry-weather system, plus the potential for data base compilation on a continuous basis, may provide benefits equal to those of wet-weather control.

The integration of computerized inline storage with dry-weather facilities has the potential to alter the operation and performance of the dry-weather treatment plant. Inline storage could control flows, reducing the influence of diurnal flows on the sewage treatment plant, and could contain emergency spills in the system before they reach the plant or enter the receiving water. The reduction of peak flows could increase treatment efficiency and reduce the effect of slug loadings to the treatment plant. This integration of controls may also reduce the need for treatment plant expansion.

System maintenance and repairs using CATAD for flow control is indicative of the multiuse benefits. Total system flow to a sewage treatment plant can be significantly reduced to allow for maintenance and repair of the interceptors and the treatment plant. Maintenance and repairs of long duration at a treatment plant can be implemented by reducing peak flows, therefore allowing total units to be taken out of service. Repairs requiring a short time and low flows can be accomplished by accentuating the diurnal flow pattern.

Metro Seattle has used flow control extensively for maintenance and repair in the sewers. Reducing the flow and virtually isolating sections of sewer have enabled repair crews to enter the isolated sections for TV inspection. This extensive program of flow control for TV inspection and repair of sewers was started in February 1977 and is ongoing at the time of this report.

## Environmental Impacts

Improvement of receiving water quality has been the main objective of the efforts by Metro and the City of Seattle. The effects of Metro's inline storage and Seattle's sewer separation have been substantial, with approximately 98% of the municipal sewage previously entering Lake Washington now being intercepted, thus reducing the nutrient loading which was a main cause of euthrophication [6]. A majority of the raw sewage previously entering Puget Sound has also been intercepted and regulated. Coliform levels have been reduced in Elliott Bay by 63% to 98% [13]. Impacts on the receiving waters surrounding Seattle have been [9]:

- A 78% reduction of peak loading of nitrite-nitrate nitrogen
- A 81% reduction of peak loading of phosphate
- A 80% reduction of COD loading for total storm rainfalls between 0.025 cm (0.01 in.) and 6.9 cm (2.72 in.)
- A 85% reduction of peak solids load
- An increase from 2.5 mg/L to 4.5 mg/L of dissolved oxygen in the Duwamish River [13]

A total reduction of loadings to receiving waters has resulted in a rapid increase in transparency measurements in Elliott Bay and Lake Washington. Transparency readings during 1970, the year Seattle's inline storage was activated, indicate an average increase from about 1.8 m (6 ft) to 5.5 m (18 ft) [13].

During the period of implementation of both the inline storage system and sewer separation, and the dry-weather collection and treatment facilities, fish catches from surrounding waters have increased. This indicates a general water quality improvement that is attributable, in part, to the overall wastewater control program in Seattle [13]. Trawl catches of the English sole population and adult salmon returns are shown in Table 102.

Table 102. SUMMARY OF FISH INCREASES FROM 1967 THROUGH 1970 [13]

| | Adult salmon returns (Green River hatchery) | | Trawl catches of English sole[a] | | |
|---|---|---|---|---|---|
| Year | Chinook salmon | Coho salmon | 1st Ave. So. | 16th Ave. So. | Station KW |
| 1967 | 5,030 | 12,736 | 8.7 | 7.0 | 0.4 |
| 1968 | 8,114 | 50,856 | 15.5 | 2.4 | 0.4 |
| 1969 | 6,650 | 36,000 | 10.9 | 9.2 | 3.9 |
| 1970 | 9,000 | 70,868 | 32.6 | 50.2 | 11.7 |

a. Average catch per trawl at sampling locations.

## Socioeconomic Impacts

### Public Acceptance--

Concern for aesthetics and public use has been a major force behind the public acceptance of major projects to correct water pollution problems. Because the receiving waters in the Seattle metropolitan area are used for commercial and recreational fishing, swimming, boating, bathing beaches, and serve commercial marinas and seaplane businesses, the deterioration of water quality was quite noticeable. Before correction of these problems, raw sewage and combined sewer discharges were responsible for high turbidity, odors, unsightly floatables, and rapid eutrophication. The public was not only offended by these unpleasant conditions, but public beaches and swimming areas were consistently closed for public safety reasons due to high coliform counts.

Aesthetics and public use have also been major considerations in the correction of the problem. Treatment plants are landscaped and have shrubs, trees, and flower gardens to enhance their appearance, and regulator stations in heavily populated areas have received special architectural design. The Dexter Avenue regulator station serves as a bus stop shelter, and the Denny Way regulator station is incorporated into a landscaped waterfront public park, as shown in Figure 89.

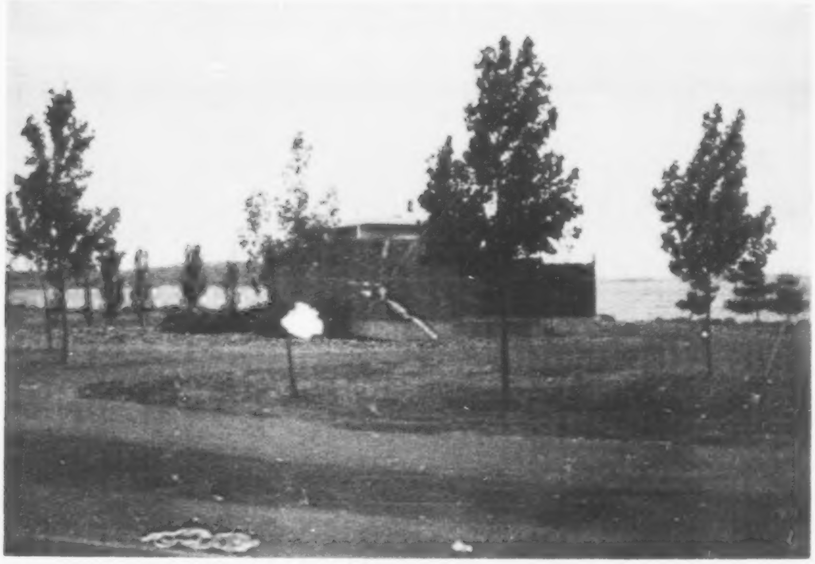


Figure 89. Denny Way regulator station/landscaped park.

Impacts on Local Community--

Impacts on the local community resulting from the areawide combined sewer overflow controls have never been quantified and in some areas are subjective. Before the sewer separation program, many private homes and commercial buildings suffered from periodic surcharging of sewers. This problem has not been eliminated, but has been reduced substantially. The correction of the situation has eliminated financial loss by the property owners and financial loss by the city and Metro because of a reduced number of lawsuits.

Waterfront businesses and recreation areas were affected by the deterioration of the surrounding receiving waters, but the impacts on land values or tax assessments have not been quantified. Considering that 80% of the city border is water [14] and 60% of the shoreline is public parks [1], the adverse impacts on the local community were significant enough to attack the combined sewer overflow problem.

Displacement or inconvenience of people or facilities was limited primarily to the construction of regulator stations and associated facilities. The temporary inconveniences to the public would be those associated with construction, including noise, increased air pollution, and traffic disruption. However, the inline storage solution resulted in minimal disruptions when compared with the potential inconvenience of sewer separation construction in high density commercial or business areas.

Impacts on Future Facilities

Inline storage, because of its flexibility of operation, can provide a foundation for future system expansion or addition of facilities when a higher level of combined sewer overflow control is needed. Using or expanding inline storage offers one of the most cost-effective control options available. Costs of offline storage, for example, can range from $0.16/L ($0.62/gal) of storage capacity for simple open earth structures to over $1.98/L ($7.50/gal) for complex concrete storage/sedimentation facilities [10], when compared with less than $0.18/L ($0.69/gal) for inline storage.

The ability to modify the operation of the system is also valuable to any facility because future needs are not always predictable. Seattle's computer augmented inline storage has used some aspects of its potential modified operations to allow extensive inline repair work to be completed with minimal system disruption. Long-term modifications can be made at minimal expense by reprogramming the computer for permanent changes in the system. For example, a large increase in population could require reprogramming of the computer to reflect decreased detention times due to increased flow.

The Seattle 201 study includes plans to increase storage capacity by using offline storage to meet the desired reduction of overflows to 10 per year [15, 16]. The offline storage units would be adaptable to the present control system, and CATAD can be modified to control offline storage as well as inline storage. Projected costs and annual overflow volume reductions for future system expansions are shown in Figure 90.

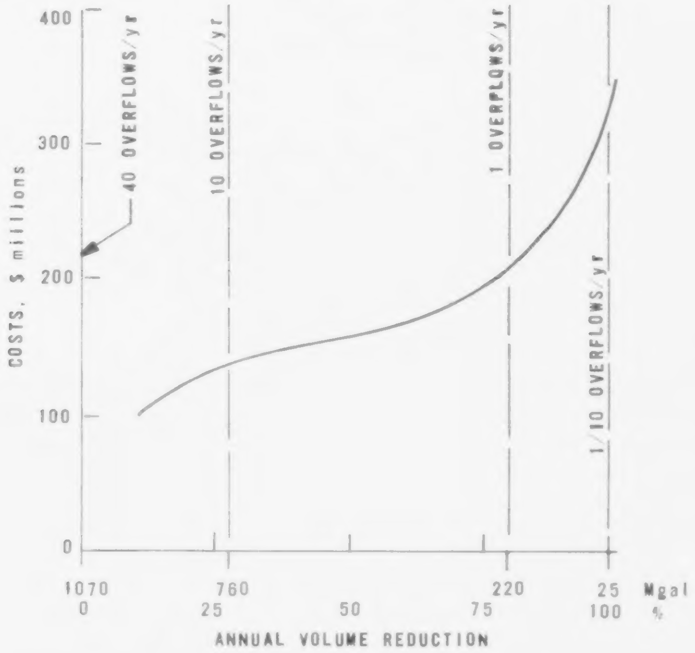


Figure 90. Estimated costs and overflow volume reduction for future system expansion, Seattle [16].

## Section 13

# CONTROL OF COMBINED SEWER OVERFLOWS USING STORAGE/SEDIMENTATION - SAGINAW, MICHIGAN

Controlling combined sewer overflows from metropolitan areas can be achieved by a combination of storage and treatment processes that take advantage of the existing system characteristics and capacities and require relatively little land area for construction of the treatment system. Storage and treatment by sedimentation both reduce the volume of overflow to receiving waters and provide treatment approaching primary levels for all overflows. Saginaw's Hancock Street facilities are an example of the use of inline storage and offline storage/treatment as a part of a citywide plan to eliminate uncontrolled combined sewer overflows to the Saginaw River.

The Hancock Street facilities consist of an integrated system of inline storage, using existing interceptor capacity controlled by modified regulator stations; a flood protection pumping station; and a storage/treatment basin capable of treating and disinfecting all overflows. The basin exhibits a high suspended solids concentration removal efficiency, over 70%, on the treated overflows. Removals of BOD and heavy metals are also high, ranging between 40 and 60%. The storage/treatment basin design is similar to that of New York's Spring Creek [1] and Boston's Cottage Farm [2] facilities.

The storage/treatment basin was designed for multiuse, with the construction of a two-level parking garage over the basin. This resulted in no loss of land area for community use and can be considered a socioeconomic benefit to the older central commercial district adjacent to the facilities.

A limited monitoring program was conducted at the storage/treatment facilities as a part of this case history assessment. The program afforded an opportunity to collect much needed data on the operation of this type of control system. The results were used to characterize the combined flow entering the basin, to evaluate the effectiveness of the basin in terms of hydraulic and pollutant removal performance, and to estimate the effect of the basin on loads discharged to the Saginaw River.

## CONTROL SYSTEM DEVELOPMENT AND DESCRIPTION

Saginaw is typical of most medium-sized cities with combined sewer systems that experience periodic discharge of combined sewage to nearby receiving waters. Concern over water quality in the Saginaw River has prompted the development of a control plan to reduce or eliminate the combined sewage loads originating from the developed urban areas. The plan includes an integrated combination of the use of existing facilities and structural solutions to achieve an optimized control system.

Area Characteristics

Saginaw is in the eastern part of central Michigan north of Detroit, as shown in Figure 91. The Saginaw River is the principal receiving water flowing through the middle of the urban area to Lake Huron. The topography of the area is flat, and the elevation is approximately 180 m (600 ft).

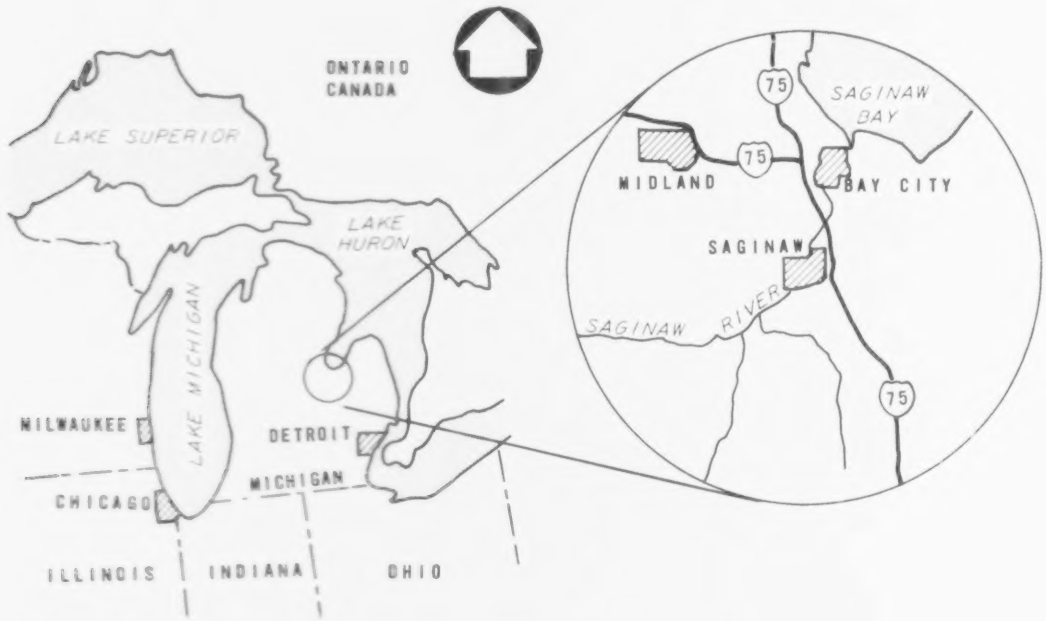


Figure 91. Saginaw, Michigan.

The area, which is surrounded by the Great Lakes, has a quasi-marine environment. Temperatures stay warmer later in the year and cooler during the spring because of the influence of the large water masses.

Saginaw's normal annual precipitation is about 70 cm (28.5 in.), over 50% of which falls during the summer between May and September. The monthly distribution of rainfall is shown in Figure 92. The summer storms occur as showers and thunderstorms. Most of the winter precipitation is snow, which accumulates until the spring thaw. Beginning in March or April, melting snow combined with rainfall can produce high overflow volumes and increased discharge of pollutant loads to the river.

Most of the storms are of low average intensity. Over 90% of the time, the average intensity of rainfall that produces runoff is less than 0.25 cm/h (0.10 in./h), as shown in Figure 93. Approximately 525 hours of rainfall producing runoff occurs annually [3].

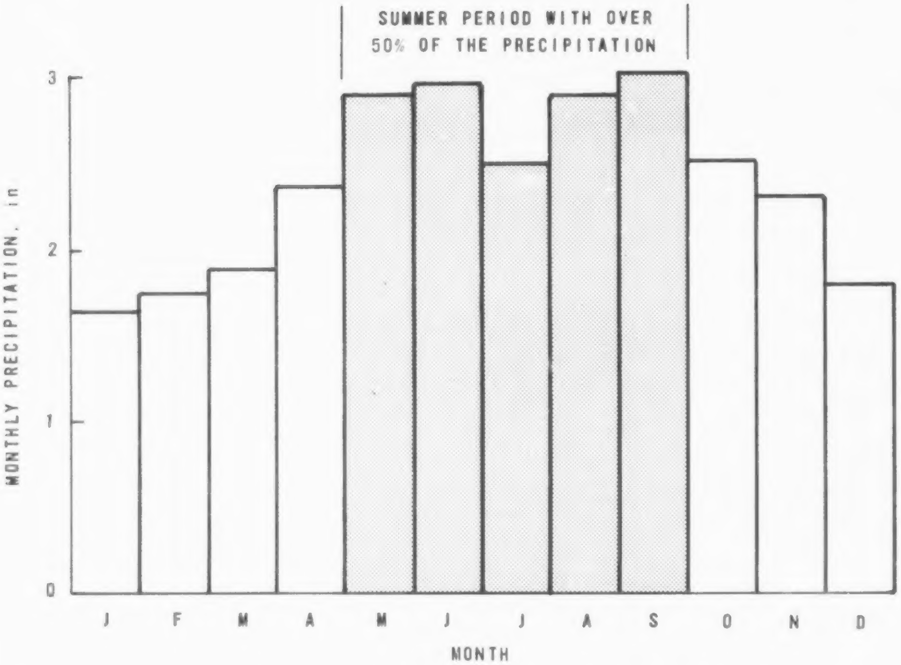


Figure 92. Normal distribution of annual precipitation, Saginaw.

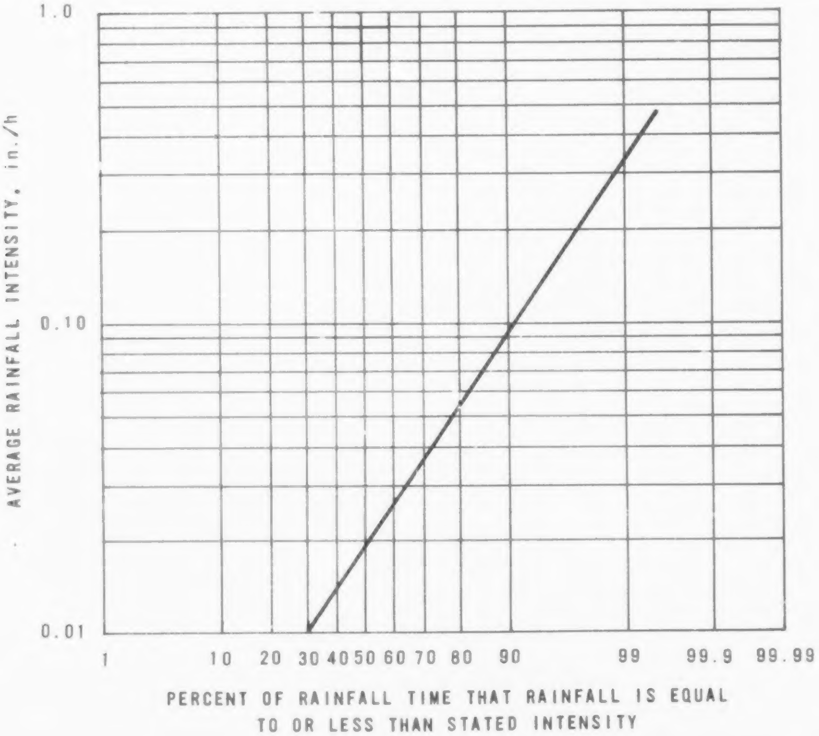


Figure 93. Probability of occurrence of rainfall producing runoff [3].

The area in Saginaw tributary to the combined sewer system is approximately 4,100 ha (10,200 acres), most of which is fully developed commercial and residential land uses. The average imperviousness of the area is between 45 and 50% [4].

## Problem Assessment

During periods of rainfall, combined sewage overflowed the system at 34 regulator overflow points along the east and west banks of the Saginaw River. Overflows occurred when the flow in the combined collection system was greater than two to three times the dry-weather flowrate. It was estimated that approximately 60 overflows occurred per year [3].

### Impacts on the Saginaw River--

Impacts on the Saginaw River from combined sewer overflows included dissolved oxygen depletion in the downstream segments of the river and increased bacteria levels.

The Stormwater Management Model (SWMM) was used as a design tool to project the expected suspended solids and $BOD_5$ loads to the river resulting from a storm of 4.8 cm (1.9 in.) of rainfall. This storm represents a 1 year storm frequency. The model was calibrated on a small, 243 ha (600 acre), subarea of the city and was then used to project the loading from a larger subarea. Loadings from the city's 4,100 ha (10,200 acre) total area were interpolated from these results. The citywide 1 year storm loadings were estimated at 18,200 kg (40,000 lb) of $BOD_5$ and 182,000 kg (400,000 lb) of suspended solids [4].

The estimated impact of these loads, together with the continuous point discharge of treated sewage effluent, could result in a dissolved oxygen level in the river of less than 4 mg/L at a minimum daily river flow of 17.4 $m^3/s$ (615 $ft^3/s$). At the minimum monthly river flow of 24.8 $m^3/s$ (874 $ft^3/s$), the resulting river dissolved oxygen was estimated at between 5 and 6 mg/L [4].

Public health impacts to the river could also be significant because of the high level of microorganisms associated with combined sewer overflows. Suspended solids and floatable material also create visual problems and accentuate public awareness of the health of the river.

### Characterization of Combined Sewage--

The results of the limited monitoring program conducted at the Hancock Street storage/treatment facilities were used to characterize the influent combined sewage. It was assumed that this influent combined flow was representative of the overflow quality.

The average suspended solids concentration of the combined flow during the summer of 1978 (May through September) was about 400 mg/L, and about 110 mg/L for $BOD_5$. More important, however, was the change in pollutant concentration as a function of time since the beginning of an overflow. Transient suspended solids and BOD concentrations of combined flow are shown in Figures 94 and 95. The plotted values represent discrete sample data. Confidence limits of 10 and 90% were determined to show the range of variability and to show the trend toward decreasing concentration as the overflow progresses. Suspended solids concentration extremes ranged from a 10% value of 175 mg/L to 1,050 mg/L at 90% during the first 0.5 hour of flow. The similar range for $BOD_5$ was 40 mg/L and 355 mg/L.

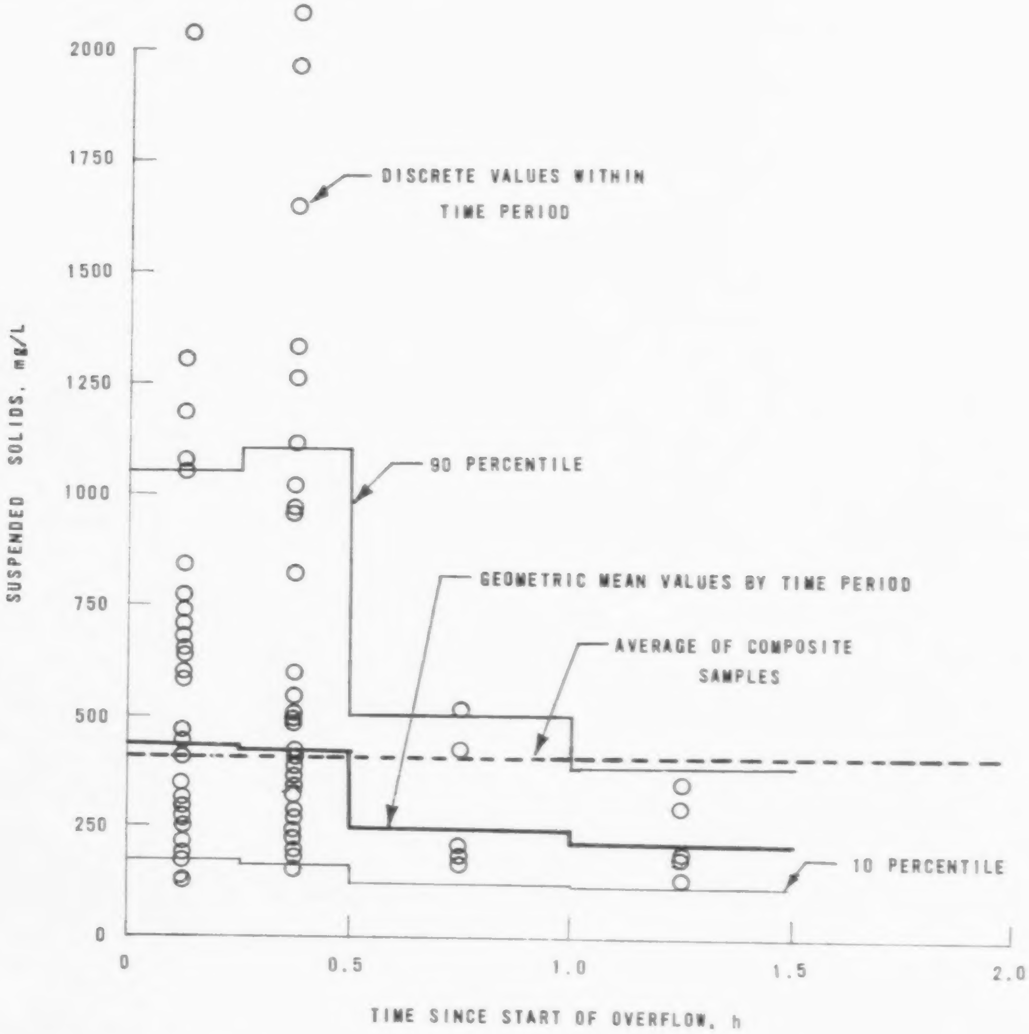


Figure 94. Characterization of combined overflow suspended solids, Hancock Street storage/treatment facilities.

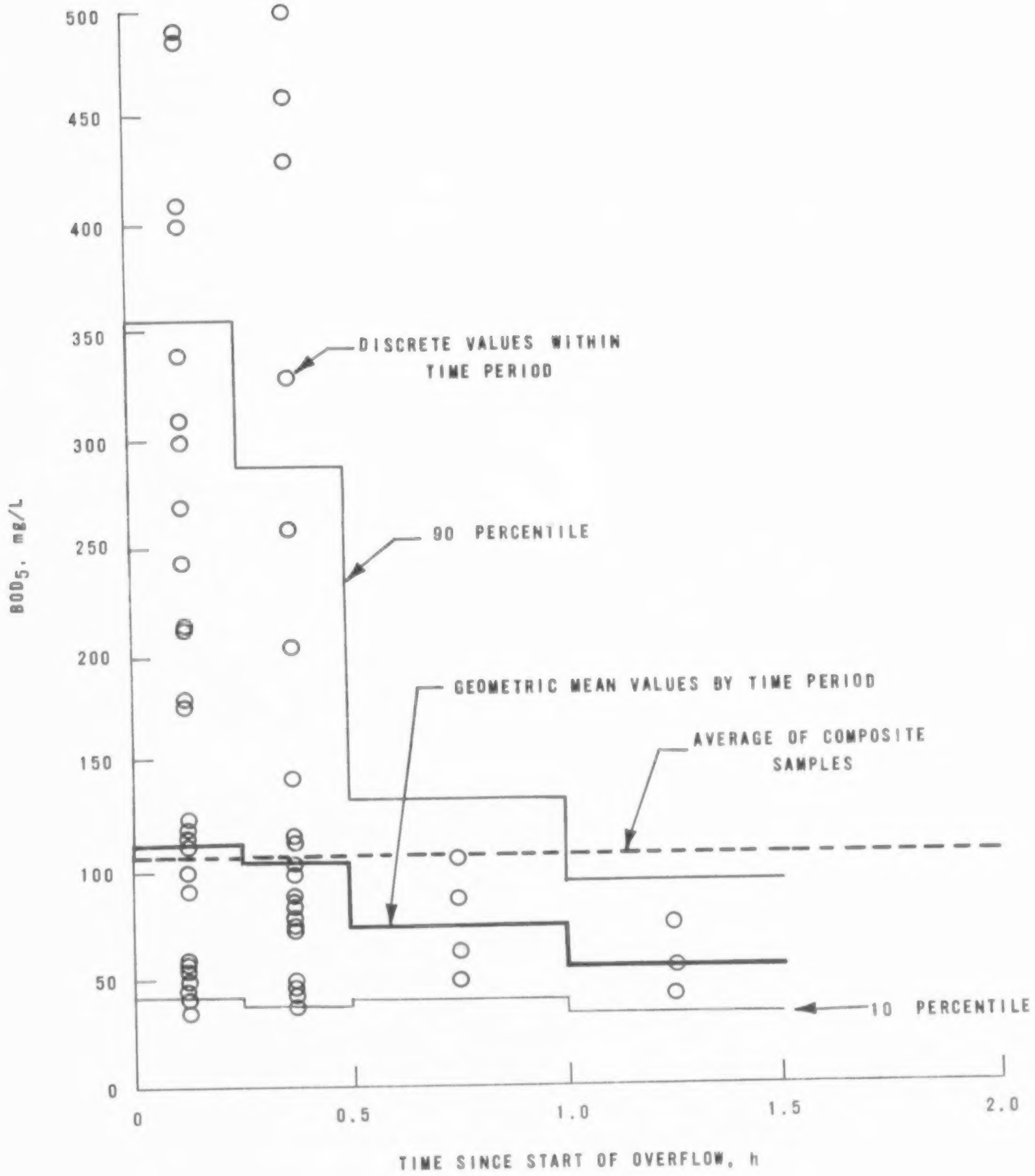


Figure 95. Characterization of combined overflow BOD$_5$, Hancock Street storage/treatment facilities.

Other characteristics of the combined sewage are presented in Table 103 for nutrients and heavy metals. These values represent composite samples of the inflow to the storage basin. Approximately 53% of the suspended solids were inorganic. Fecal coliform values ranged from 1.6 x $10^4$/100 mL to over 6.0 x $10^7$/100 mL. The average pH of the flow was 7.2 and ranged from 6.7 to 7.7.

Table 103. QUALITY CHARACTERISTICS OF COMBINED SEWER FLOWS ENTERING THE HANCOCK STORAGE/TREATMENT FACILITIES

| | Constituent | | | | | |
|---|---|---|---|---|---|---|
| Parameter | COD, mg/L | Total nitrogen, mg/L | Total phosphorus, mg/L | Lead, μg/L | Total chromium, μg/L | Zinc, μg/L |
| Average[a] | 392 | 5.9 | 2.8 | 118 | 32 | 602 |
| 90 Percentile | 1,028 | 9.9 | 5.1 | 206 | 67 | 1,595 |
| 10 Percentile | 149 | 3.5 | 1.5 | 67 | 15 | 227 |

a. Geometric average of 7 to 12 samples, each.

Pollutant Loadings--

Because of the configuration of the regulators and the available inline storage of the collection system, an estimated 50% of the combined sewer load was treated at the dry-weather treatment facilities. The other half was discharged to the river during overflow events. Suspended solids loading to the river has been estimated at about 1.8 to 2.3 million kg (4 to 5 million lb). The $BOD_5$ loading was estimated at approximately one-fourth of the suspended solids load.

During the monitoring program, an estimated 200,000 $m^3$ (52 Mgal) of combined flow was pumped to the storage facility. Assuming this volume represents that which would have overflowed, a suspended solids load of 82,000 kg (180,000 lb) and a $BOD_5$ load of 23,000 kg (50,000 lb) would have been discharged to the river. These loads correspond to a measured total rainfall, for the storms monitored, of 15.8 cm (6.2 in.).

## Recommended Plan to Control Combined Sewer Overflows

A plan to control combined sewer overflows was developed with the following objectives: maximizing the use of the existing system, protecting receiving water quality, treating and disinfecting all overflows, and preserving the function of the existing flood protection system. The plan included modifying the existing regulator structures to optimize the use of inline storage capacity and constructing seven storage/treatment basins.

The plan was developed from modeling, using the Stormwater Management Model (SWMM), and statistical evaluation of rainfall/frequency data to evaluate the most cost-effective combination of storage and treatment. The sizing of storage to contain the first 1.3 cm (0.5 in.) of runoff was determined to be sufficient for achieving the following goals of the planned system:

- Reduce the 60 raw overflows to 15 treated overflows.
- Provide the equivalent of primary treatment for the 1 year storm (design event) in the storage/sedimentation basins.
- Provide the equivalent of secondary treatment on an annual basis by capturing and treating the combined sewage overflows, i.e., achieve approximately an 80% suspended solids and BOD mass removal.

The total system storage to be provided was estimated at about 140,000 $m^3$ (37 Mgal) and was split about equally between inline and offline storage. The system was to operate automatically during a storm event, with no major mechanical equipment in the basins.

As a part of the overall plan, the Hancock Street storage/treatment facilities were constructed. None of the remaining six basins have been constructed at present. A schematic of the existing facilities (34 regulators and 5 flood pumping stations), the proposed combined sewer treatment facilities, and the Hancock Street facilities are shown in Figure 96. Based on the large percentage of small volume rainfall/runoff events, maximum use of the inline system was considered an important criterion to reduce the offline treatment capacity required downstream.

## Hancock Street Storage/Treatment Facilities

The Hancock Street facilities serve about 650 ha (1,600 acres) of the combined sewered area. The control facilities are shown in Figure 97.

### Inline Storage--

The inline storage potential of the combined sewers was increased by modifying the existing float-operated regulators in the system. Before modification, the combined sewers could store a significant amount of excess flows, but due to the arrangement of the floats, the regulators were imbalanced. Modifications were made to more accurately proportion the maximum dry-weather flow capacity, which also improved the use of inline storage capacity for storing combined sewage.

Modifications to the regulators included replacing the tide gates at the outfall with motor-operated sluice gates controlled by an ultrasonic level sensing device. A regulator is shown in Figure 98. Before modification, only dry-weather flows and some wet-weather flows were diverted to the interceptor during normal river stages; now, all flows are detained and diverted to the interceptor. The estimated total developable inline storage volume in the Hancock Street system is about 16,100 $m^3$ (4.26 Mgal); however, only about 25% of about 4,000 $m^3$ (1.07 Mgal) has been developed [5]. Most of this is attributed to the modified regulators.

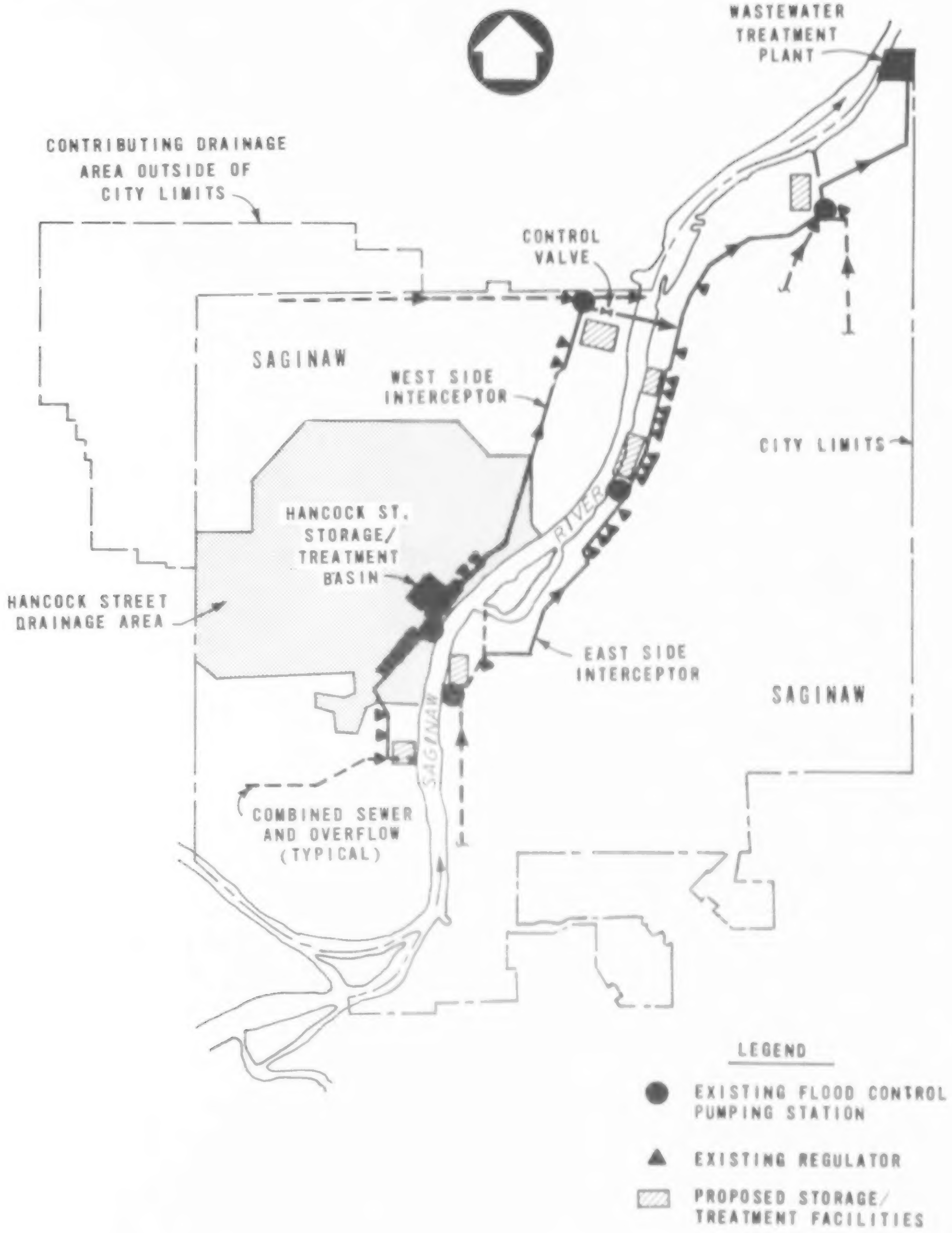

WASTEWATER TREATMENT PLANT
CONTRIBUTING DRAINAGE AREA OUTSIDE OF CITY LIMITS
CONTROL VALVE
SAGINAW
WEST SIDE INTERCEPTOR
CITY LIMITS
HANCOCK ST. STORAGE/ TREATMENT BASIN
HANCOCK STREET DRAINAGE AREA
RIVER
SAGINAW
EAST SIDE INTERCEPTOR
SAGINAW
COMBINED SEWER AND OVERFLOW (TYPICAL)
LEGEND
● EXISTING FLOOD CONTROL PUMPING STATION
▲ EXISTING REGULATOR
▨ PROPOSED STORAGE/ TREATMENT FACILITIES

Figure 96. Schematic of the proposed combined sewer overflow system and the Hancock Street facilities, Saginaw.

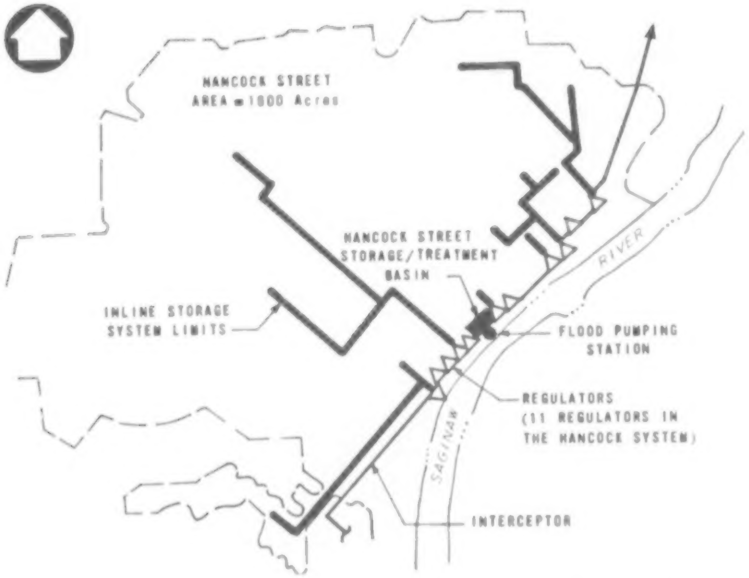


Figure 97. Components of the Hancock Street combined sewer overflow control facilities.

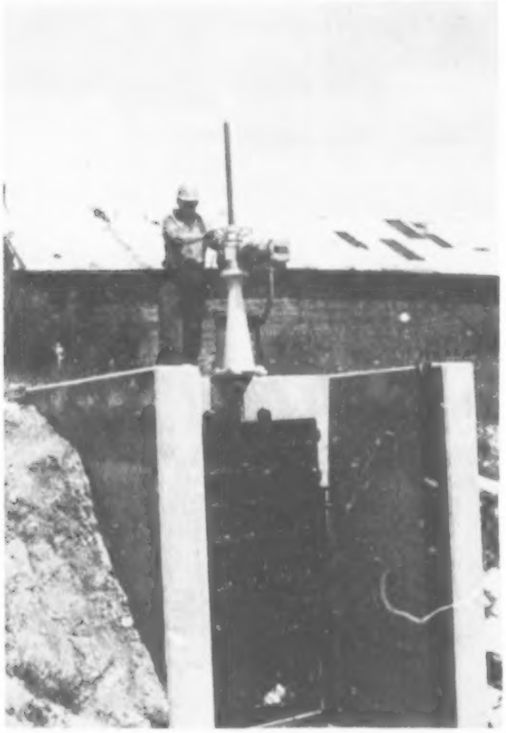


Figure 98. Regulator station with motor-operated sluice gate.

Flood Pumping Station--

During storms, combined flows diverted to the interceptor by the regulators activate the pumping station as the interceptor level increases. Sluice gates between the interceptor and the pumping station wet well open automatically and pumping is sequentially controlled by sensing the water elevations in the wet well. All flows to the storage/treatment basin are pumped. The pumping station facilities are shown in Figure 99.

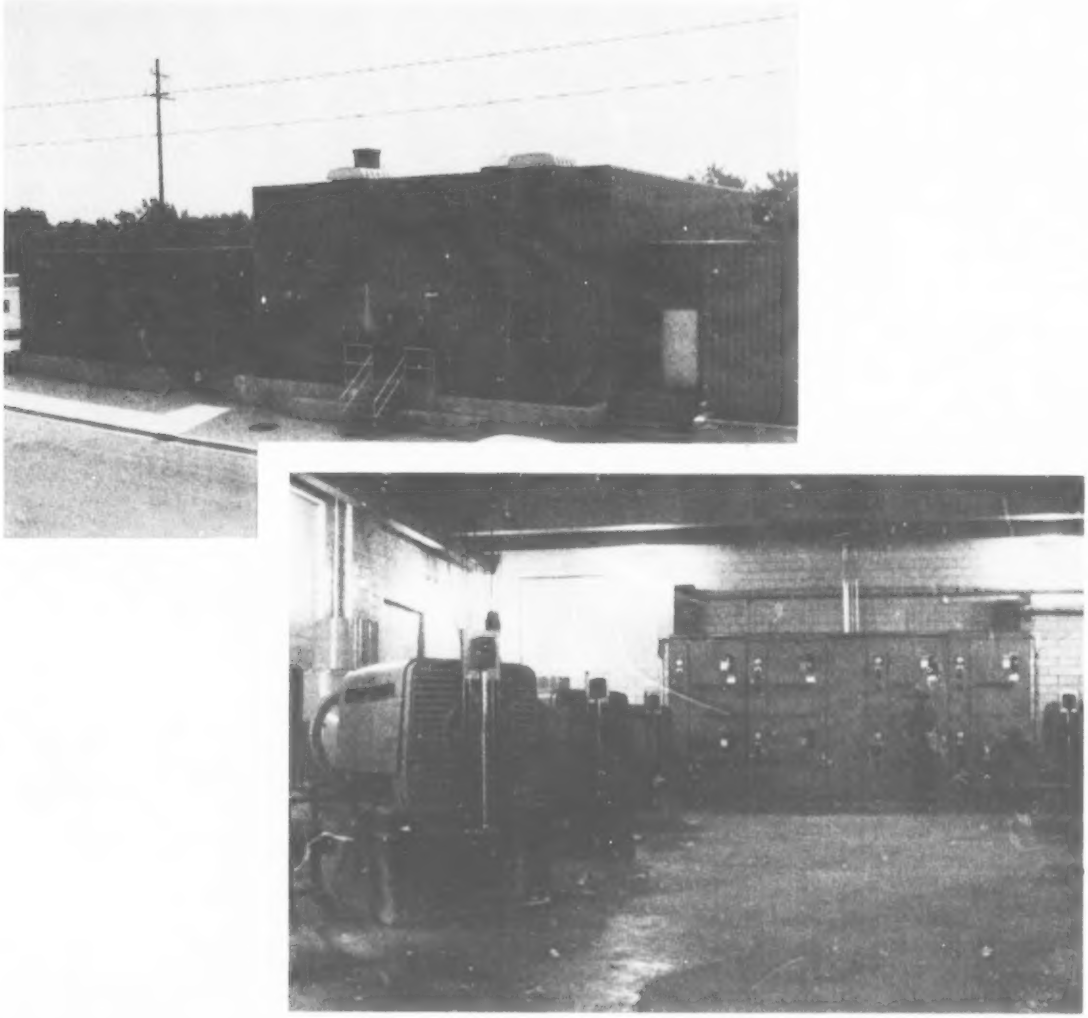


Figure 99. Hancock Street flood control and combined sewage pumping station.

During extreme river flooding, the system will revert back to operation as a flood control pumping station, with flows to the basin being diverted to the river once the system has been filled to capacity.

Disinfection facilities, flushing water pumps, sampling equipment, a new control system, and personnel facilities were added to the existing flood pumping station when it was integrated into the combined sewage treatment system.

Storage/Treatment Facilities--

Combined flows pumped to the storage/treatment facilities sequentially fill a series of paired basins. The facilities operate under two types of storm flow conditions:

1. For certain low volume storms, the flow will be captured and totally stored, with subsequent release to the interceptor as capacity becomes available.

2. For storms producing flow that exceeds the basin storage capacity, the system will store, treat the combined flow by sedimentation, and disinfect the overflow before discharge to the river.

A flow schematic of the storage/treatment basin operation is shown in Figure 100. Flow is pumped to the basins through an influent conduit and is first distributed to Bay 2. When Bay 2 is approximately 60% full, Bay 1 will start filling. Sequential filling and isolation of Bay 2 allows the first flush loads to be captured in one bay and reduces the potential maintenance and cleanup operations for small storms. Floating oil and scum baffles in Bays 1 and 2 are provided to trap floatable material.

Depending on the magnitude of the storm volume, flow enters a transfer channel for distribution to the last two bays, which fill sequentially. When the bays are full, flow will pass through the entire basin system, and the basin will act as a settling tank. Before discharge to the effluent channel, the flows pass up through No. 3 mesh screens located between the effluent weirs and the effluent scum baffle to capture any floatable and suspended material not removed in the settling bays. The Hancock Street storage/treatment facilities are shown in Figure 101.

Disinfection Facilities--

The combined flows entering the basin are disinfected at the pumping station using sodium hypochlorite. The hypochlorite feed system is activated when the main pumps are activated and is paced by the flow into the basin, measured by the number of pumps in operation at the time. The storage/treatment basin provides the chlorine contact time before discharge of the overflow to the river.

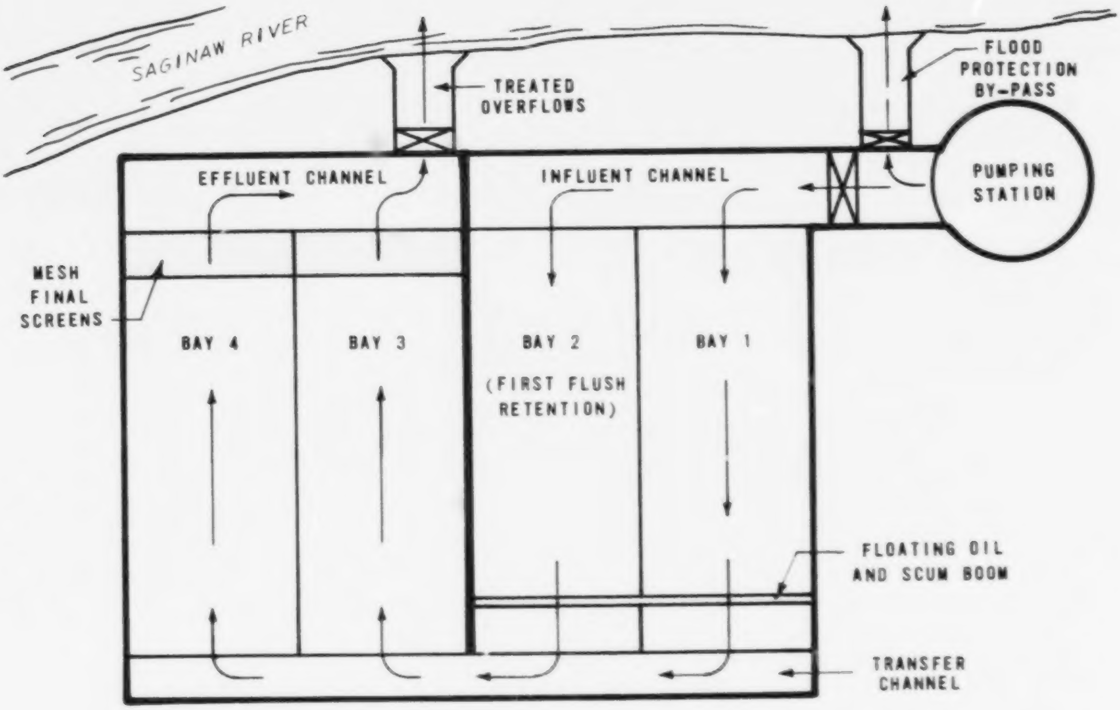


Figure 100. Flow schematic of the Hancock Street storage/treatment facilities.

The system takes dilution water from the river and injects it into the influent conduit to the basin. The hypochlorite feed pumps discharge into the dilution water pump discharge line. The system also can add hypochlorite directly to the storage basin through the flushing system headers.

The hypochlorite is purchased commercially, diluted to about 5% solution, and stored in two 60,000 L (16,000 gal) storage tanks. Dilution of the hypochlorite to 5% prolongs the effective life of the solution. The disinfection facilities are shown in Figure 102.

Dewatering and Cleaning--

After a storm, the basin is dewatered back to the interceptor, as capacity at the dry-weather treatment plant becomes available. The bays are dewatered one at a time, through a separate drain system.

The bottoms of the bays have sloped floors, 1:12, to a central flushing channel, which is sloped the length of the bays. A flushing system of high-pressure, high-volume flushing nozzles, is located on the walls of the bays. In addition, four high-pressure, manually operated monitor nozzles are mounted

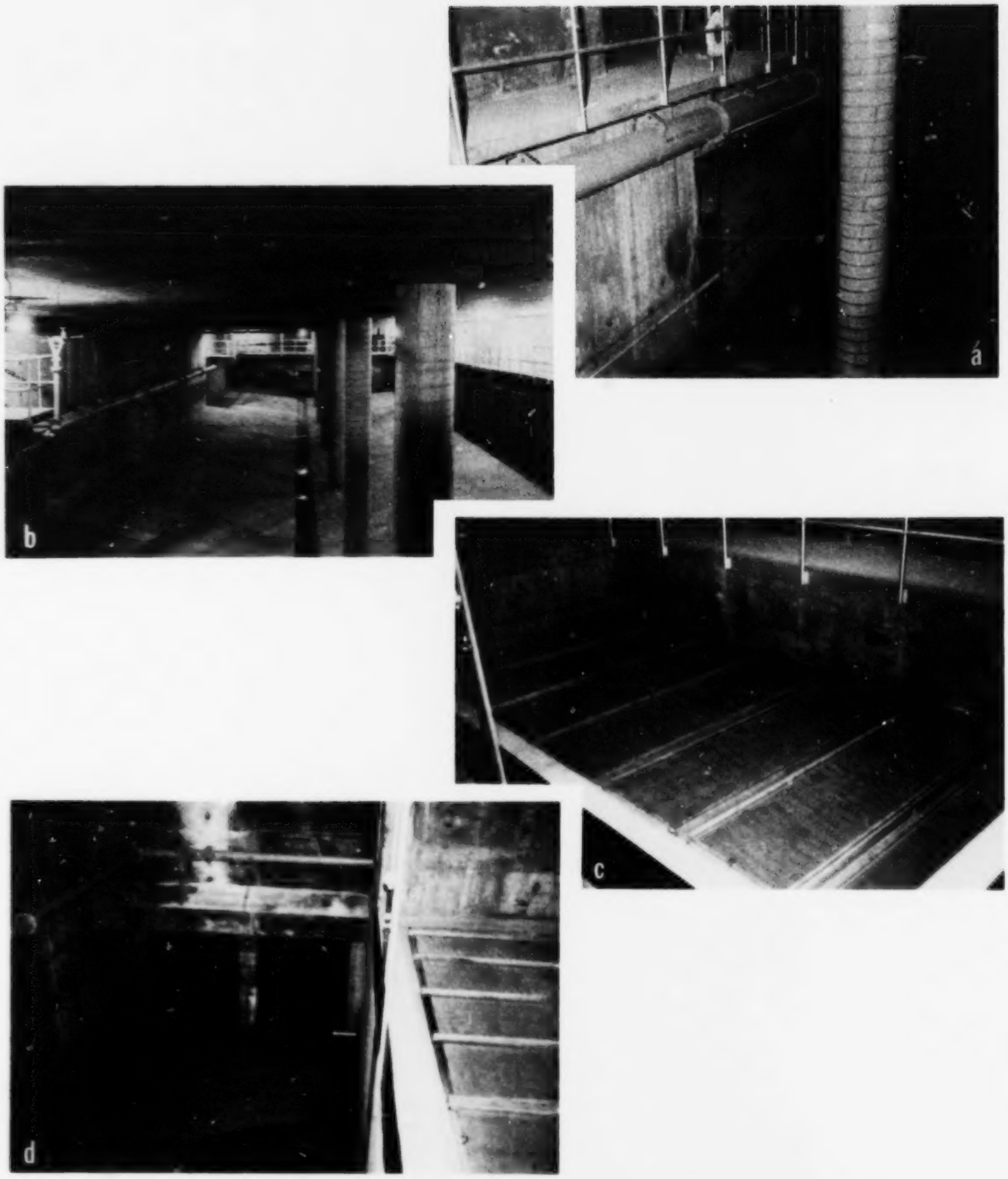


Figure 101. Hancock Street storage/treatment facilities: (a) influent orifice in Bay 1; (b) storage/treatment basin, Bay 3; (c) final effluent screens; and (d) effluent screens and effluent channel control gates.

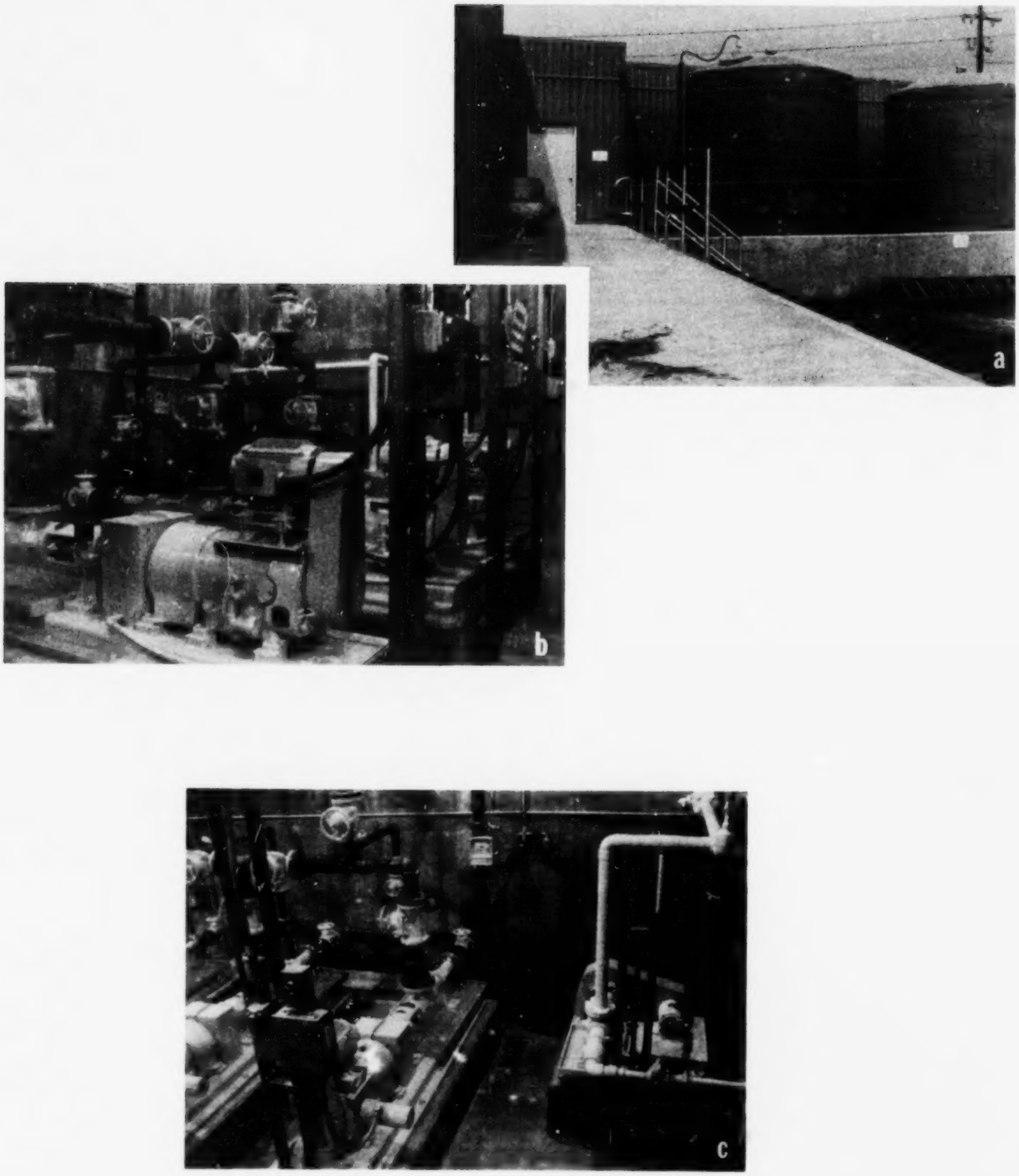


Figure 102. Hypochlorite feed system: (a) two 16,000 gal hypochlorite tanks; (b) and (c) hypochlorite pumping facilities using positive displacement pumps.

at points along each bay for further washing of the walls and cleanup of the bottoms. Flushing water is drawn from the river and pumped to the basin.

The basin, after storing several storms, was exceptionally clean, indicating that the flushing system works effectively. The flushing system components are shown in Figure 103.

## Design Criteria

The design parameters of the Hancock Street system are presented in Table 104. Parameters include design flows, storage and treatment system sizes and rates, and disinfection system criteria.

Table 104. HANCOCK STREET STORAGE/TREATMENT DESIGN PARAMETERS [5]

| Parameter | Value |
|---|---|
| Design flows | |
| Design storm frequency, yr | 1 |
| Total accumulated rainfall, in. | 1.9 |
| Peak design flow, Mgal/d | 323 |
| Peak hydraulic capacity (eight 65 Mgal/d pumps), Mgal/d | 520 |
| Storage/treatment system | |
| Inline storage | |
| Volume, Mgal | 4.3 |
| Median volumetric displacement time, min | 30 |
| Storage/sedimentation basin (4 bays) | |
| Volume, Mgal[a] | 3.5 |
| Length (per bay), ft | 206.3 |
| Width (per bay), ft | 51.8 |
| Depth (average), ft[b] | 10 |
| Floor side slope, in./ft | 1.0 |
| Central drain slope, in./ft | 0.2 |
| Surface area (total of all 4 bays), $ft^2$ | 42,700 |
| Detention time at peak design flow, min | 15 |
| Median volumetric displacement time, min | 23 |
| Peak hydraulic loading rate at peak design flow, $gal/ft^2 \cdot d$ | 8,100 |
| Disinfection | |
| Chemical | Liquid sodium hypochlorite at 5% |
| Storage tank volume (2 at 16,000), gal | 32,000 |
| Dosage rate, available chlorine at 4.5%[c] | |
| Design peak flow, mg/L | 12 |
| Range, mg/L | 6 - 25 |
| Design feed capacity, lb $Cl_2$/h | 1,200 |

[a] Includes influent and effluent conduit storage volume.

[b] At high river flows, depth in basin may increase, therefore the storage volume would increase.

[c] Projected hypochlorite strength after 90 days of storage.

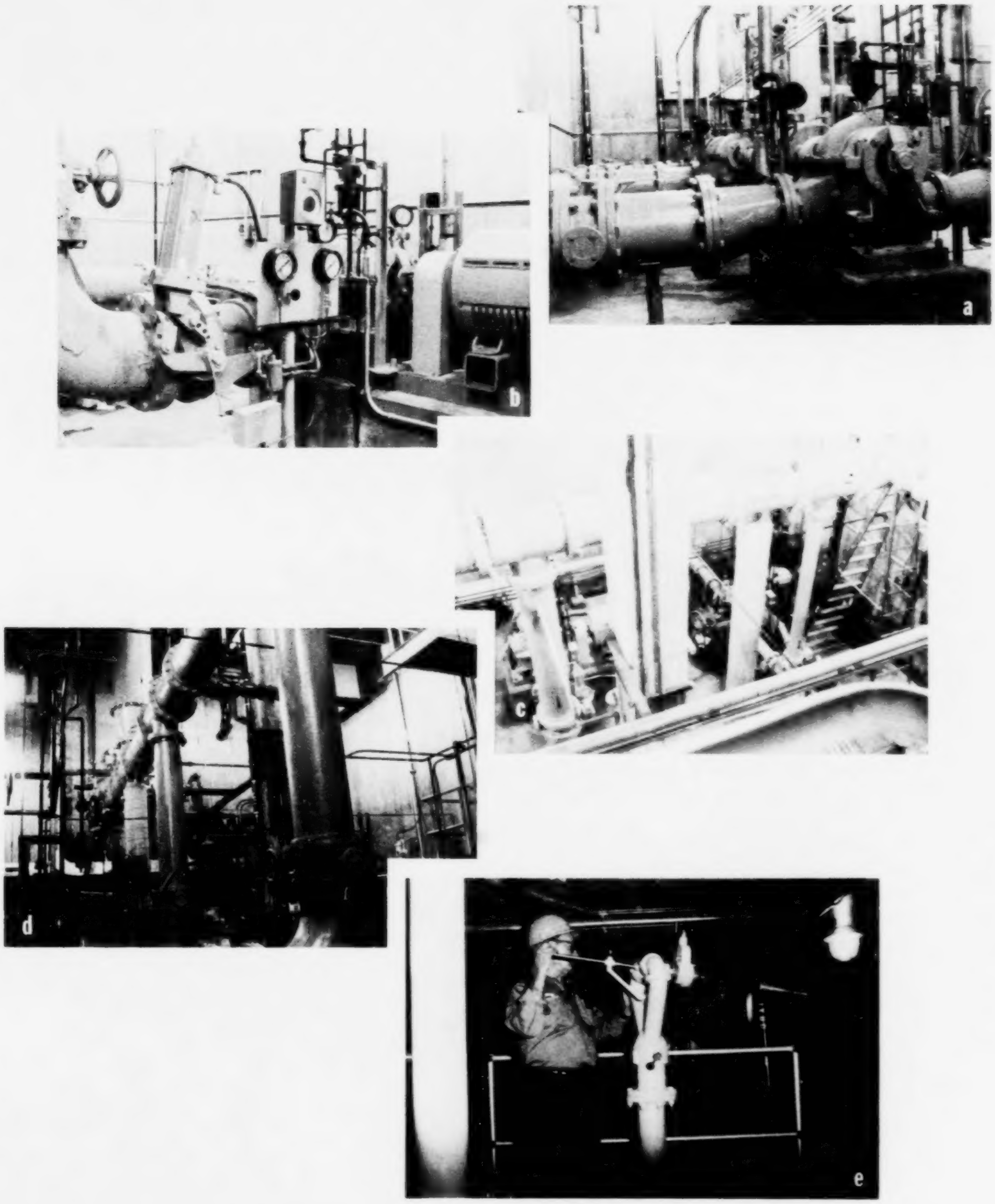


Figure 103. Hancock Street flushing system: (a) high-pressure flushing water pumps; (b) control valve on pump discharge; (c) and (d) flushing water manifold and piping; and (e) manual monitor nozzle station in the basin.

The Hancock Street basins are covered and special design considerations were used. These included basin access, placement of equipment in the basin, and ventilation requirements.

Personnel access is an important safety feature in basins that are covered, buried, or have enclosed areas subject to potential flooding. In the Hancock Street basin, the maximum distance between any two access points is 37 m (120 ft). In addition, access hatches are provided for the removal of equipment, such as motor-operated gates and screens. A system of walkways is provided for routine maintenance and inspection and access is also provided for the removal of grit accumulations that cannot be flushed out with the flushing system.

Mechanical and electrical equipment are placed at levels high enough so that during surcharged or flood conditions the equipment will not be damaged by water.

A ventilation system capable of 12 complete air changes per hour on a continuous basis is also provided. Charcoal filters deodorize the exhaust air from the basin.

## PERFORMANCE AND OPERATION

The performance of the Hancock Street storage/treatment facilities is characterized using a limited number of collected data from the monitoring program. Although this information is not considered statistically significant, it does indicate the level of effectiveness of both the storage/treatment facilities and the integration of these facilities with dry-weather treatment.

The pumping and the disinfection systems are key mechanical facilities in the overall operation of the basin. Initial startup problems and operational problems with these systems have been experienced.

### Storage/Treatment Performance

The evaluation of the Hancock Street storage/treatment facilities' performance includes analysis of:

- Overflow frequency and volume reduction
- Pollutant concentration reduction
- Pollutant load reduction

These performance evaluations should be weighed against the projected citywide combined sewage control system performance and the expected effectiveness of the Hancock Street facilities.

A 75% overflow frequency reduction was projected for the citywide system of storage/treatment facilities, storing an estimated 1.27 cm (0.5 in.) of runoff. The overall pollutant load removal by these facilities has been estimated at 92% for suspended solids and 90% for BOD [4].

The projected performance of the Hancock Street treatment efficiency at the design storm of 4.83 cm (1.9 in.) of rainfall was 49% for suspended solids and 32% for BOD. From analysis of rainfall and storm occurrences, an overflow frequency reduction of about 70% would be expected during the summer storm period [3].

Overflow Frequency and Volume Reduction--

During the summer monitoring period, 3 of the 11 storm events exceeded the Hancock Street storage capacity and resulted in overflow. The overflow frequency reduction effectiveness over this period was 73%.

The estimated total volume pumped to the storage/treatment basin during the 11 storm events was about 200 ML (52 Mgal), with about 79 ML (21 Mgal) of the total being treated and discharged to the river. Most of the overflow volume, 50 ML (13 Mgal), occurred during one storm event with a total of 3.43 cm (1.35 in.) of rainfall. The other two storm events causing overflow were less severe, with overflows of 6 and 23 ML (1.6 and 6.1 Mgal), as shown in Table 105.

Table 105. SUMMARY OF STORM AND BASIN ACTIVATION EVENTS DURING THE HANCOCK STREET MONITORING PERIOD

| Date | Average precipitation, in.[a] | Average intensity, in./h[b] | Volume pumped to basin, Mgal[c] | Overflow volume treated and discharged, Mgal |
|---|---|---|---|---|
| 5/12/78 | 0.21 | 0.11 | 2.62 | 0 |
| 5/30/78 | 0.68 | 0.14 | 4.19 | 0 |
| 6/12/78 | 0.44 | 0.19 | 3.60 | 0 |
| 7/21/78 | 0.42 | 0.15 | 1.08 | 0 |
| 8/16/78 | 0.50 | 0.29 | 3.70 | 0 |
| 8/19/78 | 0.52 | 0.15 | 5.12 | 1.60 |
| 9/12/78 | 0.48 | 0.11 | 3.68 | 0 |
| 9/13/78 | 1.35 | 0.17 | 16.79 | 13.27 |
| 9/17/78 | 0.32 | 0.13 | 0.90 | 0 |
| 9/20/78 | 0.90 | 0.28 | 9.66 | 6.14 |
| 9/27/78 | 0.39 | 0.11 | 0.89 | 0 |
| Total | 6.21 | | 52.23 | 21.01 |

a. Average of two rain gage measurements.

b. Average rainfall divided by average duration.

c. Starting on 8/19/78, volumes determined from pump operation logs. All previous values were estimated by water level observations.

Assuming that without the storage facilties all pumped flows to the basin would have overflowed, the effectiveness of the basin in reducing overflow volume is 60%. This overflow reduction is for the basin only and does not include the inline storage effectiveness. Using an assumed inline system volume reduction effectiveness of 50%, the effectiveness of the overall system, inline and offline storage/treatment, may approach 80%.

Pollutant Concentration Reduction--

When the capacity of the storage/treatment basin is exceeded, the system treats the overflows by sedimentation before discharge to the river. During the three overflow events, the basin effectiveness averaged 73% for suspended solids and 54% for BOD. The flows to the basin produced a large range of average hydraulic overflow rates in the bays; from rates approaching a secondary clarifier rate, 1.65 $m^3/m^2 \cdot h$ (970 $gal/ft^2 \cdot d$), to over 3.85 $m^3/m^2 \cdot h$ (2,270 $gal/ft^2 \cdot d$). The influent concentrations and individual storm treatment efficiencies also varied, as summarized in Table 106.

Table 106. PERFORMANCE OF THE HANCOCK STREET SEDIMENTATION BASIN

| | | | Pollutant removal[a] | | | | | |
|---|---|---|---|---|---|---|---|---|
| | | | Suspended solids | | | BOD | | |
| Storm date | Avg hydraulic overflow rate, $gal/ft^2 \cdot d$ | Peak hydraulic overflow rate, $gal/ft^2 \cdot d$ | Influent, mg/L | Effluent, mg/L | Removal % | Influent, mg/L | Effluent, mg/L | Removal, % |
| 8/19/78 | 970 | 1,500 | 896 | 62 | 93 | 126 | 40 | 68 |
| 9/13/78 | 1,235 | 6,500 | 149 | 27 | 82 | 62 | 20 | 68 |
| 9/20/78 | 2,270 | 6,300 | 420 | 232 | 45 | 42 | 31 | 26 |
| Average | | | | | 73 | | | 54 |

[a] From flow-weighted composite samples of influent and effluent.

Heavy metals (including lead, chromium, and zinc), COD, volatile suspended solids, total phosphorus, and total nitrogen were also analyzed for both the influent and effluent flows from the basin. The removal efficiency of the basin for these pollutants is shown in Table 107.

Table 107. SUMMARY OF HEAVY METAL AND OTHER POLLUTANT REMOVALS FROM THE HANCOCK STREET SEDIMENTATION BASIN

| | Removal, % | | | | | | |
|---|---|---|---|---|---|---|---|
| Storm date | Lead | Chromium | Zinc | COD | Volatile SS | Total phosphorus | Total nitrogen |
| 8/19/78 | 75 | 80 | 83 | 90 | 91 | 50 | -- |
| 9/13/78 | 70 | 70 | 40 | 47 | 79 | 14 | 0 |
| 9/20/78 | 7 | 56 | 10 | 22 | 48 | 41 | 33 |
| Average | 51 | 69 | 44 | 53 | 73 | 35 | -- |

Pollutant Load Reduction--

Estimates of the pollutant load reduction by the Hancock Street storage/treatment basin alone, and more importantly, the integrated Hancock Street dry-weather treatment facilities were made using the pumped flow and average characterization from the 11 storm events. The combined sewage storage/treatment step is an important part of the treatment of combined flows; however, the dry-weather treatment process must also be considered. Both the stored volumes from small storms and the stored volumes plus solids from the treated overflow from large storms are drained back to the dry-weather plant as capacity becomes available.

Total loads to the Hancock Street storage/treatment basin of 81,250 kg (179,000 lb) suspended solids and 21,800 kg (48,000 lb) BOD were estimated from (1) the total pumped volume to the basin, and (2) the average flow-weighted quality of the influent combined flows (410 mg/L for suspended solids and 110 mg/L for BOD).

The overflow loads discharged to the river from the storage/sedimentation basin were computed using (1) the influent combined flow quality; (2) the average treatment effectiveness of the sedimentation basins (73% for suspended solids and 54% for BOD); and (3) the volume of overflow exceeding the basin storage capacity.

The efficiency of the dry-weather treatment plant was assumed to be 90% for both suspended solids and BOD.

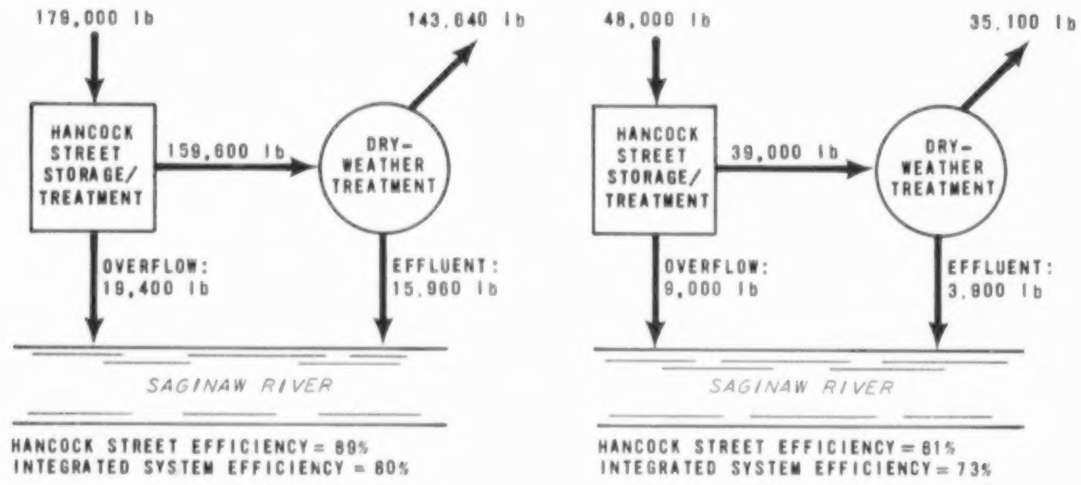
The effectiveness of the Hancock Street facilities alone was 89% suspended solids load reduction and 81% BOD load reduction from the influent flows. However, with the transport of the solids to the dry-weather treatment plant and the assumed treatment rate, the effectiveness of the total integrated system drops to 80% for suspended solids and 73% for BOD, as shown in Figure 104. Total loads to the river from the storm events are 16,050 kg (35,360 lb) suspended solids and 5,860 kg (12,900 lb) BOD.

## Operation

Although the Hancock Street system is relatively free of mechanical equipment and is a self-activating facility requiring little operator attention, the mechanical systems used (pumping and disinfection) are critical to the proper operation of the facility. Several operational problems have been identified as a reference for future design applications.

Pumping--

Initial startup problems were encountered with the main pumping controls and with the inlet sluice gates to the pumping station, and the facilities failed to properly come online for the first four or five storms. The pumping station inlet sluice gates were designed to open when the interceptor level rose to a predetermined level and to close at a lower level. Interceptor level was sensed by a sonic level sensor, which did not provide reliable level control and caused erratic opening and closing of the inlet sluice gates. To correct these problems, the controls were modified to provide automatic opening of the inlet sluice gates only. The inlet gates are manually closed after a storm event. Although a series of minor modifications were made to the sonic level sensor to improve performance, confidence in these particular devices is low.



**A. SUSPENDED SOLIDS MASS BALANCE** **B. BOD MASS BALANCE**

Figure 104. Schematic of pollutant load reductions and process elements of the Hancock Street wet- and dry-weather integrated systems.

A sequencing pumping control system was selected because the existing pumping station has no appreciable storage volume, and the existing axial flow pumps were not suitable for variable speed control. On a rising wet well level, a pressure switch would start a pump and at the same time start a timed cycle. If the wet well level did not drop before the timed cycle ended, a second pump would start and the timed cycle would restart. On falling level, a pressure switch would stop the first pump that had started and also start a timed cycle that would stop the next pump if the liquid level remained at or below the low level. In addition, if the wet well level continued to fall after the first pump stopped, a second low low level pressure switch would immediately stop the next pump, without waiting for the first timed cycle to end, and start a timed cycle of its own to shut down succeeding pumps if the liquid level remained at or below the low low level. An emergency shutdown of the pumps was also provided to stop all operating pumps when there was no longer water in the wet well to be pumped.

When the station was placed in service, it was found that the wrong type of time delay relay had been furnished. Instead of starting (or stopping) a pump and starting a timed cycle, the controls would start the timed cycle and then start (or stop) the pump. This proved satisfactory whenever only one pump was required for operation. However, serious problems occurred when several pumps were required because all operating pumps would be shut down on the emergency low level shutdown pressure switch instead of the normal shutdown controls. This method of operation caused problems with the power supply system and resulted in short cycling of the pump. During a 16 hour period of pumping station operation, 46 pump start/stop cycles were recorded.

Design of pumping facilities, where rapidly changing flow and water levels are unavoidable, should consider methods for achieving continuous pumping. This may be accomplished by using one variable speed/flow pump in sequence with constant flow pumps or by providing pump discharge control instrumentation.

Disinfection--

Several problems were encountered with the hypochlorite feed system. Progressive cavity pumps with hydraulic variable speed drives were used to pump the hypochlorite. Problems experienced with these pumps included:

- Gravity leakage through the pumps when the pumps were not operating.
- Extremely high starting torques.

When the disinfection system was placed in service, it was found there was a gravity flow of 3.8 to 7.6 L/min (1 to 2 gal/min) when the pumps were off, even though the manufacturer indicated that there should be none. A motor-operated valve was placed on the feedline to prevent gravity flow.

Because of the infrequent use of these pumps, it was found that a "set" would develop between the pump stator and rotor, resulting in extremely high starting torques. Also, the manufacturer felt that hypochlorite solution caused the stator material to slightly swell, aggravating the set between stator and rotor. This extreme starting torque would trip a torque overload switch, preventing operation of the pump. The torque overload clutches were locked-in to permit operation of the hypochlorite pumps.

Special efforts should be taken to ensure that operation of critical treatment systems is accounted for and, where necessary, alternative systems be provided.

Sluice Gates--

Many problems were experienced with the automatic operation of sluice gates. Many of these problems can be attributed to the erratic and premature operation of the ultrasonic level sensors and to faulty manufacture and installation. Several of the gate operator stem nuts stripped during operation and faulty machining was found on many of the stems.

Because of existing conditions, several of the sluice gates had nonrising stems with the stem threads submerged in the stormwater. Rising stem sluice gates should be used, if possible, so that the stem threads will not be submerged.

## ECONOMIC AND ENVIRONMENTAL IMPACTS

Assessment of the impacts of the Hancock Street storage/treatment system shows that the combination of inline storage and offline storage/treatment is a cost-effective solution to reduce the frequency, volume, and the pollutant load of combined sewer overflows. Comparing the unit costs of inline storage and the storage/treatment basin shows that use of the excess sewer system capacity is a most attractive and economical component of the control system. However, the usable volume is limited by the physical characteristics of the sewer system.

The use of the dry-weather treatment facilities as an integrated process component of the wet-weather system for treatment of the stored flows must also be considered a part of the implemented control strategy. At Saginaw, additional capacity at the dry-weather plant has been provided to handle the peak loads from the wet-weather system. Additional capacity in the sludge dewatering system has also been provided in anticipation of the solids from combined sewer overflow events. In designing a systems approach using a dry-weather treatment plant, potential operating problems can be avoided and designed for by planning and evaluating the expected increased loads.

### Economic Impacts

Costs of the inline storage and the storage/treatment basin are evaluated individually, on a unit process basis, and together as a systems approach for controlling combined sewer overflows. Annual operation and maintenance costs are estimated for the combined sewer overflow facilities and the apportioned cost of the dry-weather plant operations used to treat the stored combined sewage volumes.

#### Construction Costs--

The total construction cost of the Hancock Street storage/treatment system is $7,280,000 (ENR 3000). This cost was developed from the actual construction cost and includes both the inline storage system modifications (regulator modifications) and the storage/treatment facilities. A two-story parking garage was included as a part of the basin construction at an additional cost of $490,000.

The estimated cost of the storage/treatment facilities is $6,910,000, and includes modifications to the existing flood pumping station, the storage basin, and appurtenant facilities. Based on a peak treatment flowrate of 14 $m^3/s$ (323 Mgal/d), the treatment cost is approximately $490,000/$m^3 \cdot d$ ($21,400/Mgal·d).

Costs based on the storage capacity of the system are shown in Table 108. The inline storage system costs (about $370,000) were for modification of the existing regulators to increase the usable storage capacity of the combined sewers. The storage capacity attributable to the regulator modification cost is about 4,000 $m^3$ (1.07 Mgal).

Table 108. SUMMARY OF COSTS OF THE HANCOCK STREET STORAGE/TREATMENT SYSTEM[a]

| Component | Storage capacity, Mgal | Area served, acres | Construction cost, $ | Storage cost, $/gal | Cost per acre served, $/acre |
|---|---|---|---|---|---|
| Storage/treatment facilities | 3.52 | 1,600 | 6,910,000 | 1.96 | 4,300 |
| Inline storage | 1.07 | 1,600 | 370,000 | 0.35 | 230 |
| Total system | 4.59 | 1,600 | 7,280,000 | 1.58 | 4,530 |

a. ENR 3000.

The unit cost of inline storage is about six times less than the unit cost of constructing the storage/treatment basin. Static inline control, however, can only be developed to "safe" maximum limits of available storage capacity. Further control of and use of the available storage capacity may be achieved through automatic or dynamic control of the regulators, to use the storage more efficiently, but this increases costs.

Use of inline and offline storage together can achieve higher levels of control at lower costs than offline storage alone. A comparison of the total Hancock Street system unit costs to those of storage/treatment alone indicates about a 20% cost saving.

Annual Operation and Maintenance Costs--

The estimated annual operation and maintenance cost for the storage/treatment system is about $50,000/yr. About $15,000 of the annual cost is attributed to the dry-weather treatment plant operations for pumping and solids dewatering and disposal.

Assuming that the Hancock Street facilities alone remove 363,000 kg/yr (800,000 lb/yr) suspended solids, the annual treatment cost is approximately $0.14/kg ($0.06/lb) removed, or about $0.05/$m^3$ ($185/Mgal) stored and/or treated.

## Environmental Impacts

Due to the short period of operation and limited monitoring data on the facilities, receiving water impacts or benefits can only be projected. These projections are based on the performance of the facilities and the expected load reductions from the river.

The overall load reductions to the Saginaw River, on an annual basis from the Hancock Street area, were estimated using the resulting efficiency of the storage/treatment system and the dry-weather plant performance (80% suspended solids and 73% BOD removal). Assuming influent loads of 409,000 kg (900,000 lb) suspended solids and 114,000 kg (250,000 lb) BOD per year from the Hancock Street area, the system, including dry-weather treatment, should remove about 327,000 kg (720,000 lb) suspended solids and 82,900 kg (182,000 lb) BOD.

These removed solids, however, will be collected, treated, and disposed of at the dry-weather treatment plant. The estimated increase in the total solids load to the dry-weather plant contributed by the Hancock Street facilities is about 4% annually. For citywide combined sewage control, the increase would be about 26%, but no operating problems are foreseen.

Reduction of the overflow volume, plus disinfection of the treated overflow, should greatly improve the bacterial quality of the river during storm events. Although the river is principally used for shipping, the public health aspects of the river water quality are important if the river is to be used for recreation. During extreme storm events, the probability of contamination would be higher; but on an annual basis, the number of days of bacterial quality violation should be reduced in proportion to the overflow events being totally contained, about 75%.

Aside from the specific pollutant impacts, improvement in the health and appearance of the river is considered an environmental improvement as well as a socioeconomic benefit.

## Socioeconomic Impacts

The Hancock Street storage/treatment system has a 296-stall, two-level parking garage over the storage basin, thus providing multiuse of the land area of the storage/treatment facilities. Other impacts include improved aesthetics and the potential for contributing to the reversal of the economic decline of the older business district it serves.

The storage basin/parking garage facilities, shown in Figure 105, displaced about 0.5 ha (1.2 acres) of run-down warehouses with a new aesthetically pleasing, low profile structure providing about 1.0 ha (2.4 acres) of offstreet parking. The people displaced by the facility have relocated nearby, with little apparent disruption.

The local community favored the garage construction and actively campaigned for its inclusion in the facilities plan. The parking capacity of the facilities, however, has not been used to its fullest to date, and as a result, parking meter receipts have been lower than anticipated. Plans exist to lease the lower section of the garage, thereby increasing utilization. Land values and rentals in the business district area have been increasing, and the facilities may have contributed to that increase.

No complaints of odors from the facilities have been received and most of the people using the parking are unaware of the treatment facilities below.

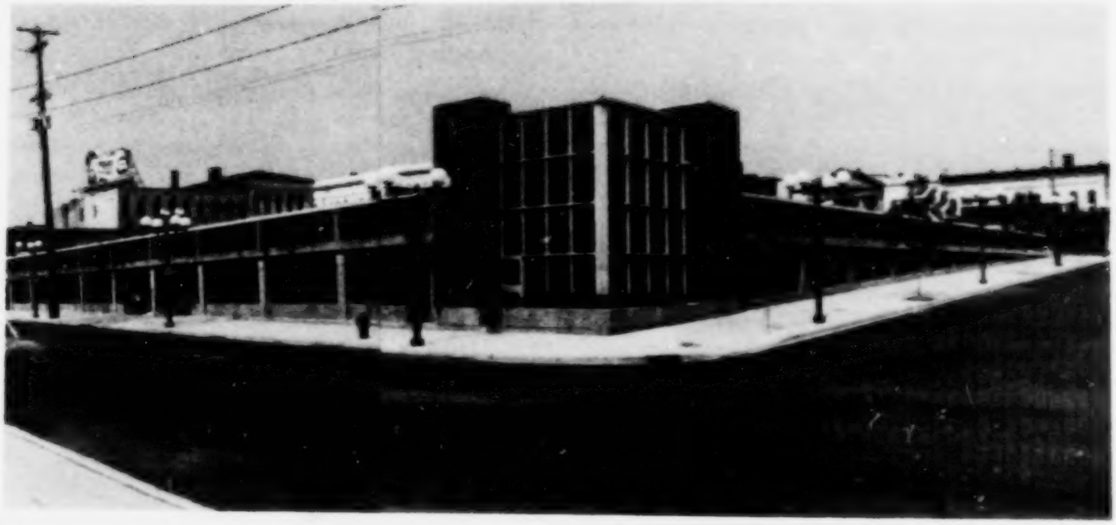


Figure 105. Two-story parking garage over the storage/treatment basin.

# SECTION 14

## MULTIUSE COMBINED SEWER OVERFLOW FACILITIES
## MOUNT CLEMENS, MICHIGAN

Combined sewer overflows have been a problem in Mount Clemens and have contributed to the degradation of the Clinton River. Approximately 87% of the city's 950 ha (2,350 acres) was serviced by combined sewers, with 24 overflow points to the river [1, 2].

Mount Clemens developed and implemented a citywide combined sewer overflow control program that included partial sewer separation, interception of most of the combined sewer overflow points with a storage/transport tunnel, and construction of combined sewage treatment facilities with a multiuse recreational park and lakelet system to polish the treated flows before discharge. The facilities are not yet operational, and it is expected that the use of the existing dry-weather treatment plant as a part of the wet-weather treatment system will not occur in the immediate future. The facilities to transport dry-weather flow to the Detroit regional treatment plant have not yet been constructed.

An EPA demonstration project was conducted in Mount Clemens to evaluate the feasibility of using physical and biological processes to treat combined sewer overflows [3]. Results of the demonstration facilities were incorporated in the design of the citywide solution.

The cost of the entire city program, including the sewer separation, is about $21.5 million, or $26,000/ha ($10,500/acre). The treatment effectiveness of the combined sewage treatment facilities is estimated at about 95% for suspended solids and BOD.

The projected impacts of these facilities include the complete elimination of untreated combined sewer overflows entering the river, although urban runoff from the separated areas will still enter the river untreated.

### PROJECT DESCRIPTION

Mount Clemens, like most cities in the Great Lakes region, originally constructed combined sewers to convey sanitary and storm flows. However, as the cities grew, the capacities of these systems were often exceeded, resulting in the raw discharge of pollutants during overflows. Mount Clemens implemented a citywide control program of storage and treatment to eliminate these overflows. In addition, certain sewers in the city were separated because it was potentially less expensive than intercepting the combined overflow points, and the treatment system uses a combination of storage and pumping to control treatment rates at the wet-weather treatment facilities.

## Area Characteristics

Mount Clemens is in southeastern Michigan about 44 km (22 mi) northeast of Detroit, as shown in Figure 106. The city has a population of about 21,000 and is located on the Clinton River about 8 km (5 mi) from Lake St. Clair [4].

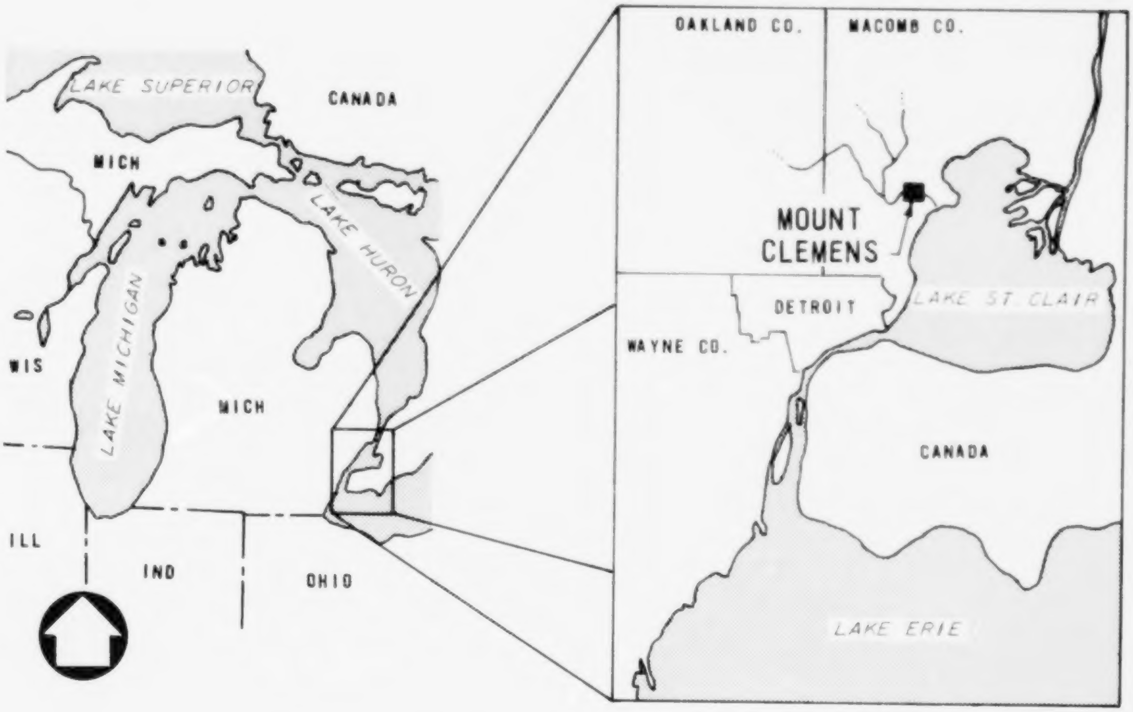


Figure 106. Mount Clemens, Michigan.

The topography is generally flat or gently rolling hills, with elevations varying between 180 m (590 ft) and 190 m (620 ft). Because of the moderating influence of the Great Lakes, the climate has gentle fluctuations with a mean annual temperature of 10°C (50°F) [3, 4]. The average annual precipitation is about 76 cm (30 in.), of which 55 to 60% occurs from April to September [5].

The land use in Mount Clemens is predominantly residential (51%). The remaining 49% consists of commercial, industrial, public and semipublic, and transportation land uses ranging from 9 to 15% each, and about 1% vacant land and water surface [2, 4]. The total area of the city served by sewers is 950 ha (2,350 acres), of which 460 ha (1,150 acres) is combined and 490 ha (1,200 acres) is separate.

## Problem Assessment

The State of Michigan identified the Clinton River as a major polluted river in the Lake Erie basin. The pollution was primarily a result of effluent discharges or combined sewer overflows or both from more than two dozen municipalities and private dischargers.

Mount Clemens, in particular, contributed significant amounts of combined sewage to the river. About 93 km (58 mi) out of 105 km (65 mi) of the city's sewers was combined [1, 3]. A storm with an intensity greater than 0.254 cm/h (0.10 in./h) would cause an overflow. An estimated 40 overflows per year discharged about 2.8 million $m^3$ (760 Mgal) of combined sewage to the Clinton River annually [1, 3, 6].

The average concentrations of $BOD_5$ and suspended solids in combined sewage in Mount Clemens were estimated to be 140 mg/L and 350 mg/L, respectively [3]. Using these concentrations and the estimated volume of combined sewage overflow, the annual combined sewage pollutant loading to the river was estimated at $4.0 \times 10^5$ kg ($8.9 \times 10^5$ lb) of $BOD_5$ and $1.0 \times 10^6$ kg ($2.2 \times 10^6$ lb) of suspended solids, or about 480 kg/ha (430 lb/acre) $BOD_5$ and 1,210 kg/ha (1,080 lb/acre) suspended solids.

Pollutant concentrations and overflow characteristics for 13 overflow events in two drainage areas in Mount Clemens are summarized in Table 109. Initial and average concentration values for suspended solids and $BOD_5$ were sampled and the data show no distinct first flush effect. These average values are consistent with the estimated citywide values.

## Implemented Countermeasure

Mount Clemens received a Demonstration Grant from the EPA to demonstrate the feasibility of treating combined sewer overflows from 86 ha (212 acres) of the city in a multipurpose physical and biological treatment facility [3]. The treatment consisted of aerated lakelets, microscreening, chlorination, and pressure sand filtration.

The demonstration project showed that the aerated lakelet system approached a 90% removal efficiency for suspended solids and BOD. About 70% of the removal occurred in the first lakelet. Intermediate algae control using microscreens had little effect on the overall performance of the system. The citywide program to eliminate combined overflows and to treat the stored combined volume was developed around the demonstration project lakelet system.

### Elimination of Overflows--

The overflows from the combined sewers in the city were eliminated by sewer separation in some areas of the city and by construction of a storage/transport tunnel to intercept the combined sewage overflow points along the Clinton River. The combined and separate sewer areas of Mount Clemens are shown on Figure 107.

## Table 109. COMBINED SEWER OVERFLOW CHARACTERISTICS OF TWO DRAINAGE AREAS IN MOUNT CLEMENS [3]

| Overflow date | Total precipi-tation, in. | Storm duration, h | Overflow volume, $ft^3$ | Overflow duration, h | Peak flow rate, $ft^3/s$ | No. of samples | Initial SS, mg/L | Average SS, mg/L | Initial $BOD_5$, mg/L | Average $BOD_5$, mg/L |
|---|---|---|---|---|---|---|---|---|---|---|
| 12/19/70 | (melting snow) | | 21,248[a] | 4.2 | 1.62 | 10 | 336 | 382 | 124 | 101 |
| 1/3-4/71 | 0.27 | 9 | 177,065[b]<br>19,730[a] | 9.1<br>5.0 | 6.08<br>0.97 | 20<br>19 | 736<br>584 | 616<br>420 | 258<br>264 | 190<br>155 |
| 2/4-5/71 | 1.32 | 12 | 215,513[b]<br>184,142[a] | 15.0<br>22.0 | 5.88<br>1.42 | 8<br>14 | 44<br>328 | 202<br>294 | 37<br>139 | 44<br>216 |
| 2/17/71 | 0.21 | 8 | 103,204[b]<br>24,283[a] | 6.1<br>4.2 | 5.24<br>1.03 | 18<br>22 | 568<br>308 | 334<br>394 | 171<br>100 | 102<br>131 |
| 2/18-19/71 | 0.18 | 6 | 115,350[b]<br>50,084[a] | 7.0<br>8.0 | 3.69<br>6.2 | 23<br>20 | 276<br>212 | 233<br>246 | 110<br>150 | 81<br>115 |
| 2/19-20/71 | 0.34 | 8 | 511,982[b]<br>237,772[a] | 16.0<br>16.0 | 5.8<br>1.16 | --<br>-- | --<br>-- | --<br>-- | --<br>-- | --<br>-- |
| 2/22/71 | 0.50 | 10 | 67,793[b]<br>24,283[a] | 7.1<br>8.7 | 6.21<br>0.58 | 19<br>17 | 368<br>340 | 516<br>548 | 59<br>32 | 46<br>52 |
| 3/6-7/71 | 0.52 | 16 | 98,145[b]<br>14,165[a] | 13.4<br>9.3 | 5.3<br>0.45 | 20<br>19 | 300<br>160 | 289<br>288 | 74<br>53 | 91<br>64 |
| 3/13/71 | (melting snow) | | 13,356[b]<br>21,046[a] | 4.0<br>3.0 | 3.43<br>1.29 | 12<br>-- | 424<br>-- | 286<br>-- | 117<br>-- | 89<br>-- |
| 3/19/71 | 0.15 | 9 | 14,166[b] | 4.7 | 3.36 | 11 | 396 | 387 | 206 | 140 |
| 5/19/71 | 0.35 | 4 | 24,284[b]<br>34,402[a] | 3.8<br>3.0 | 2.9<br>2.26 | 10<br>-- | 232<br>-- | 287<br>-- | 140<br>-- | 128<br>-- |
| 5/24/71 | 0.25 | 17 | 5,059[b] | 2.0 | 3.3 | 2 | 568 | 430 | 276 | 231 |
| 6/13/71 | 0.26 | 4 | 23,272[b] | 2.5 | 5.8 | 14 | 644 | 344 | 249 | 179 |
| Average concentration | -- | -- | -- | -- | -- | -- | 379 | 360 | 142 | 119 |

a. Spruce service district (75 acre).

b. Clemens service district (137 acre).

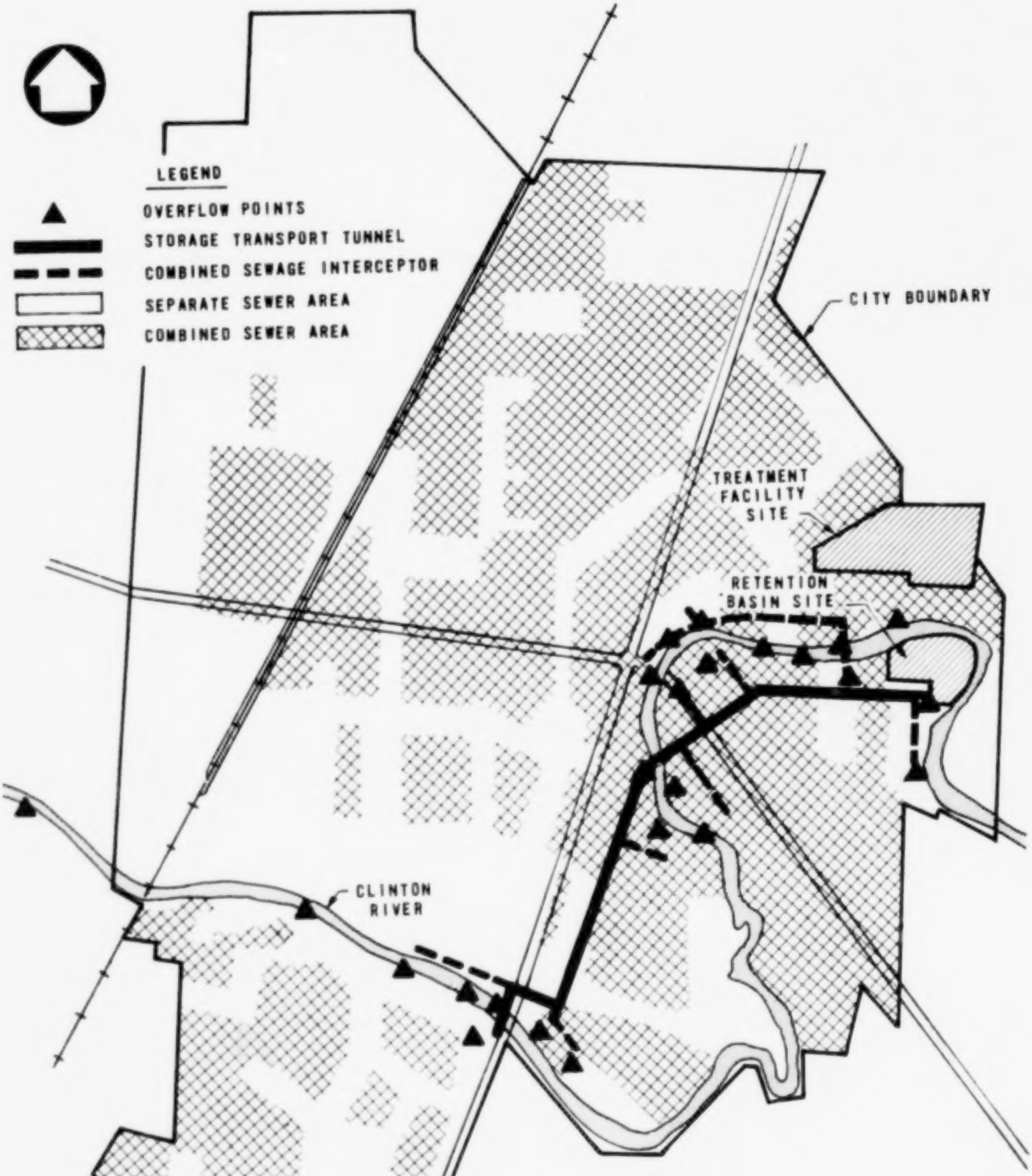


Figure 107. Mount Clemens, combined and separated sewer areas, overflow points, and control facilities.

Sewer Separation--In some areas of the city, it costs less to separate combined sewers rather than to build an interceptor for collecting combined sewage at the overflow points. Sewer separation eliminated combined sewer overflows; however, stormwater overflows still remain, and untreated urban runoff is allowed to enter the river. Suspended solids, nutrient, and toxic material problems may still persist in the river because of these flows, even though the combined flows are eliminated. The city now has approximately 490 ha (1,200 acres) of separated sewers and 460 ha (1,150 acres) of combined sewers with future plans to separate another 17 ha (41 acres) in conjunction with road repaving projects.

Overflow Interception--The storage/transport tunnel used to intercept combined sewer overflow points along the river is a major portion of the solution to eliminate discharges. The concrete tunnel stores and transports combined sewage to the treatment facility. The tunnel is 2.75 m (9 ft) square and has a storage capacity of about 12.30 $m^3$ (3.25 Mgal).

### Combined Sewage Treatment--

The combined sewage treatment facility includes storage and physical and biological treatment. The facility is located at two sites separated by the Clinton River because of land availability and system configurations. Provisions for use of the existing 11,600 $m^3$/d (3 Mgal/d) sewage treatment plant are incorporated into the design of the combined sewage treatment system. A schematic of the treatment facility components and flow diagram is shown in Figure 108. An alternative flow path bypassing the dry-weather facility is provided until sanitary flows can be sent to the regional treatment facility in Detroit and the dry-weather plant can be converted to wet-weather operation.

Retention Basin Site--The combined wastewater flows collected in the storage/tranport tunnel are pumped to sedimentation tanks to remove the grit and the heavier fraction of suspended solids. The flows from the sedimentation tanks are stored in an aerated retention basin and pumped across the river for final treatment.

A chlorination basin is incorporated in the retention basin to disinfect emergency bypass flows if the total storage capacity of the retention basin is exceeded. The chlorination basin can also store excess flows from the retention basin. Chlorination occurs only during emergency bypassing of flows, otherwise unchlorinated flows stored in the contact basin are pumped across the river to the park treatment site. Components of the retention basin site are shown in Figure 109.

Park-Treatment Site--The treatment park facility includes the existing sanitary sewage treatment plant, aerated lakelets, and sand filters. Modification of the existing sewage treatment plant will provide grit removal, clarification, and chlorination for the stored flows pumped from the retention basin site. The effluent from the modified sewage treatment plant undergoes biological treatment in the lakelet system. The series of three lakelets (the first two are aerated) are followed by sand filtration. A portion of the aerated lakelets and one of the pressure sand filters are shown in Figure 110.

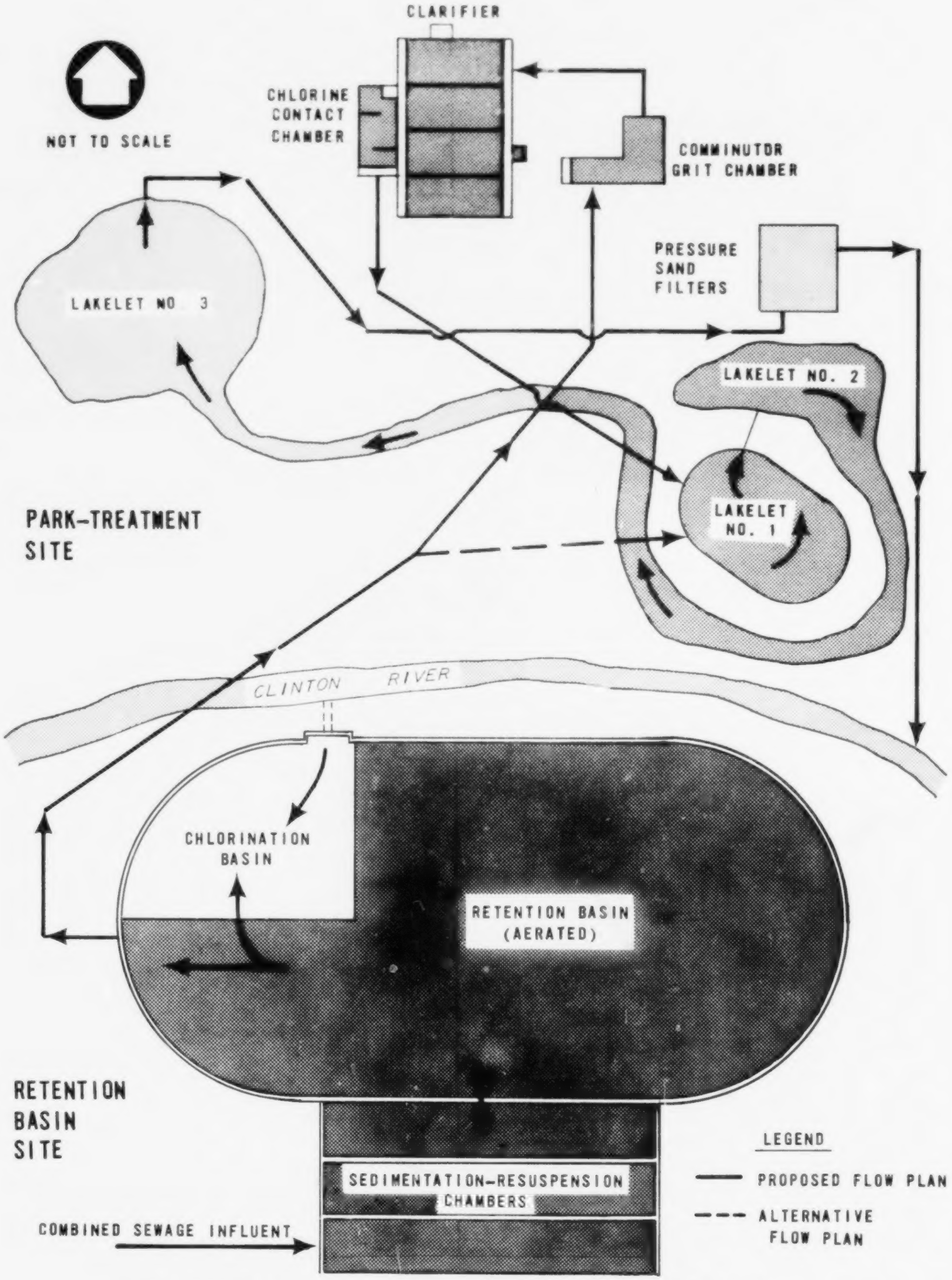


Figure 108. Schematic of Mount Clemens combined sewage treatment facilities.

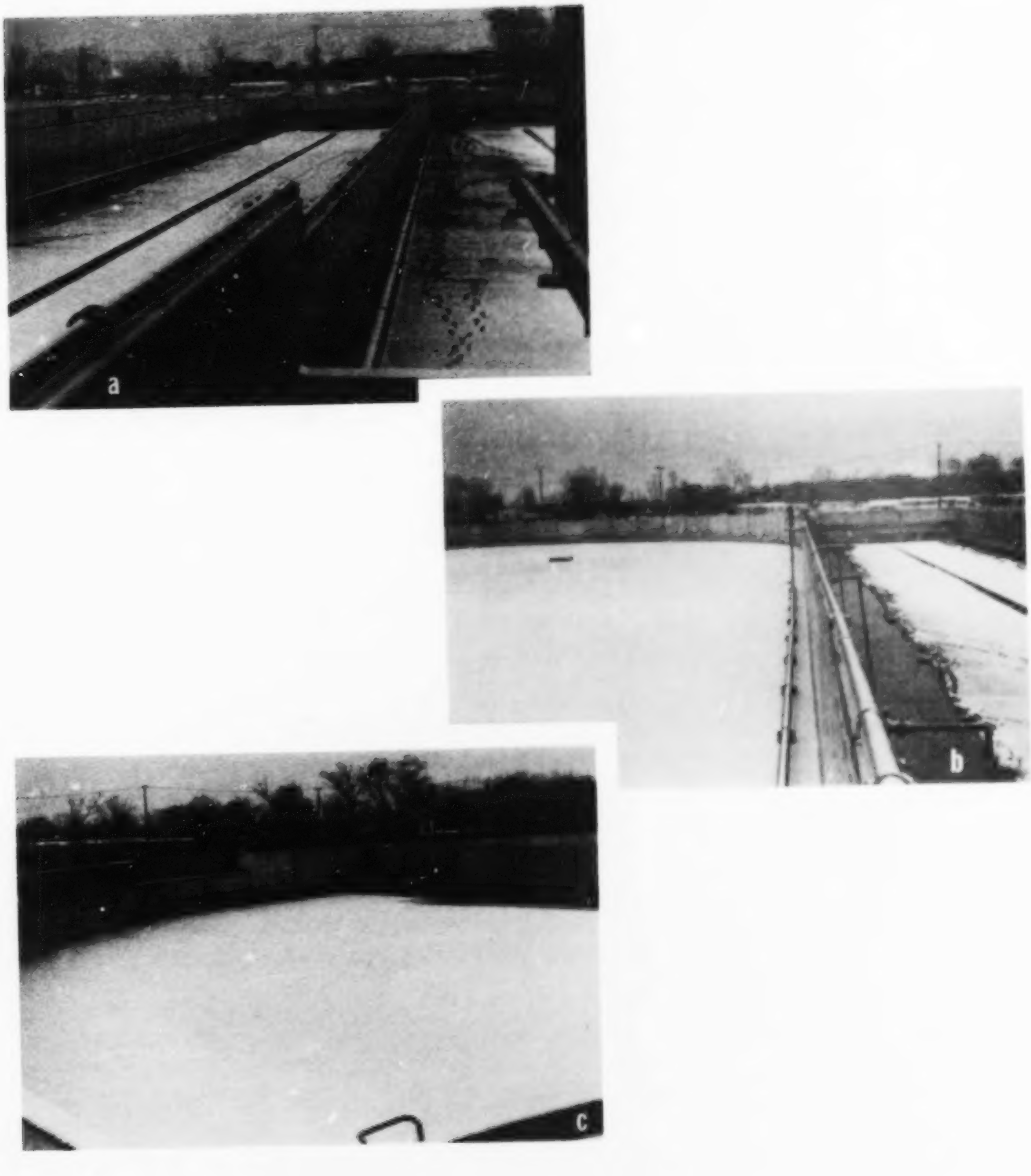


Figure 109. Mount Clemens retention basin site components: (a) sedimentation-resuspension chambers (SRC), (b) aerated retention basin, and (c) chlorination basin with overflow outlet.

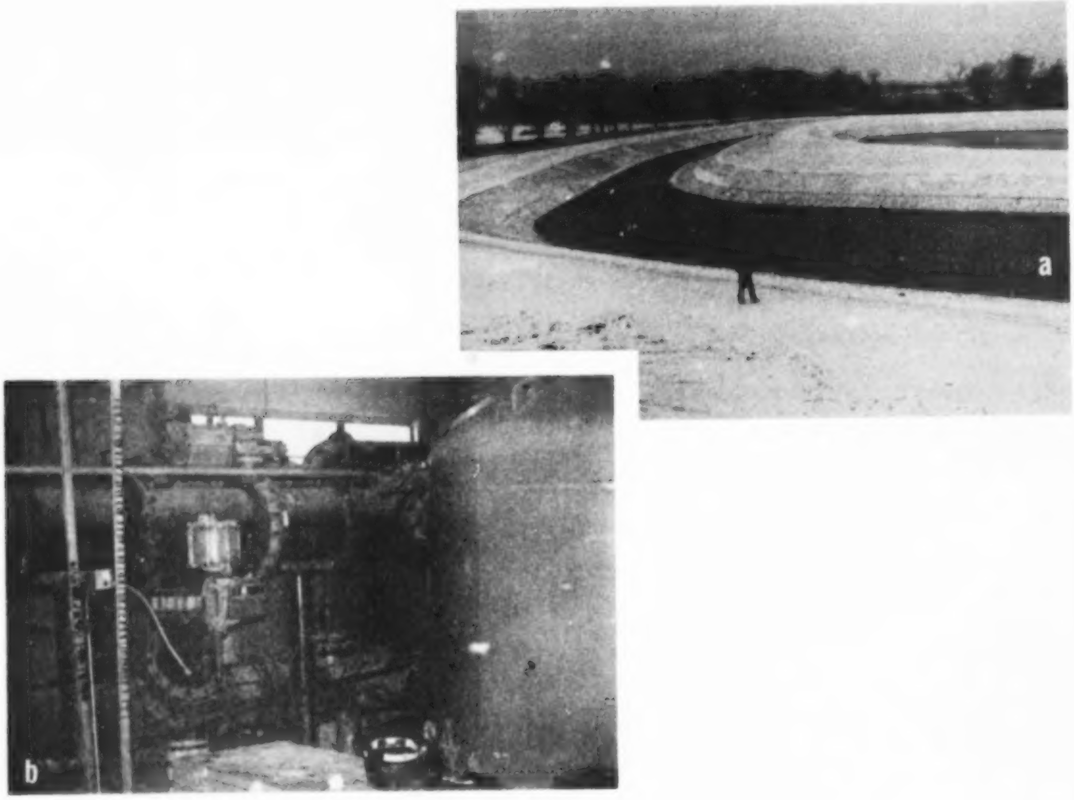


Figure 110. Park treatment facility for combined sewage: (a) a portion of the aerated lakelets, and (b) pressure sand filter.

Design

The stormwater facilities are designed to control the projected peak flow from a 5 year storm and to meet strict effluent standards in the city's discharge permit. The design incorporates a combination of storage and pumping to control flow and treatment rates through the various processes of the treatment system. The maximum pumping rate of combined sewage into the retention basin is 950,000 $m^3$/d (250 Mgal/d). The average dewatering rate to the park treatment facility is 15,000 $m^3$/d (4 Mgal/d). The criteria used for the design of the individual units of the storage and treatment processes are shown in Table 110.

Table 110. COMBINED SEWER OVERFLOW TREATMENT FACILITY DESIGN PARAMETERS, MOUNT CLEMENS [3]

| Parameters | Value |
|---|---|
| Storage/transport tunnel | |
| Size, ft | 9 x 9 |
| Capacity, ft$^3$ | 434,000 |
| Retention basin site | |
| Peak inflow rate (5 pumps at 50 Mgal/d each), Mgal/d | 250 |
| Sedimentation resuspension chamber (3 bays) | |
| Length per bay, ft | 360 |
| Width per bay, ft | 64 |
| Surface area, ft$^2$ | 23,000 |
| Peak hydraulic loading rate, gal/ft$^2$·d | 3,600 |
| Minimum hydraulic loading rate, gal/ft$^2$·d | 730 |
| Detention time at peak flow, min | 30 |
| Retention basin (includes chlorination basin) | |
| Surface area, ft$^2$ | 360,000 |
| Volume, ft$^3$ | 3,625,000 |
| Maximum dewatering rate, Mgal/d | 5 |
| Chlorination | |
| Detention time at peak flow (200 Mgal/d), min | 20 |
| Dosage rate, mg/L | 5 |
| Park-treatment site | |
| Average flowrate, Mgal/d | 4 |
| Clarifier | |
| Length, ft | 118 |
| Width, ft | 84 |
| Depth, ft | 20 |
| Surface area, ft$^2$ | 10,000 |
| Hydraulic loading rate at 4 Mgal/d, gal/ft$^2$·d | 400 |
| Detention time, hr | 3 |
| Chlorination | |
| Detention time at peak flow (5 Mgal/d), min | 15 |
| Maximum dosage rate, mg/L | 10.0 |
| Lakelet No. 1 | |
| Surface area, ft$^2$ | 52,000 |
| Volume, ft$^3$ | 500,000 |
| Detention time, d | 1 |
| Lakelet No. 2 | |
| Surface area, ft$^2$ | 122,000 |
| Volume, ft$^3$ | 1,100,000 |
| Detention time, d | 2 |
| Lakelet No. 3 | |
| Surface area, ft$^2$ | 100,000 |
| Volume, ft$^3$ | 920,000 |
| Detention time, d | 1.8 |
| Sand filtration | |
| Number of filters | 4 |
| Surface area, ft$^2$ | 50 |
| Hydraulic loading rate, gal/ft$^2$·min | 15 |

## OPERATION AND PERFORMANCE

The operation and performance of the wet-weather treatment facilites are based on the results of the demonstration project and estimates of the expected efficiency of those process elements not evaluated in the demonstration project [3]. The facilities are expected to become operational by mid-1979.

## Operation

The combined sewage treatment system can operate either (1) using modified components of the existing dry-weather treatment facilities (planned system operation) or (2) bypassing the dry-weather plant with flows being pumped directly to the lakelet system (alternate system operation). Until facilities are available to transport the city's sanitary flows to the regional treatment plant, Mount Clemens will continue to process sanitary flows through the dry-weather facilities and use the alternate system operation to handle wet-weather flows.

### Planned System Operation--

Under the planned system operation, the dry-weather facilities would be modified to treat the retention basin flow before it enters the lakelet system. Modifications include disconnecting the trickling filters and converting the primary and secondary sedimentation tanks into a wet-weather clarifier. The chlorine contact chamber following the clarifier would be used to control bacteria in the lakelet system. Solids handling operations and plant personnel requirements were established for this mode of operation.

Solids Handling--Solids handling operations include the collection and disposal of the solids removed in the wet-weather processes. Following a storm, the settled solids and grit in the sedimentation tanks are resuspended in a slurry and pumped to the modified dry-weather treatment plant. After the resuspended slurry is pumped, the stored combined flows from the retention basin are pumped to the modified dry-weather plant, then to the aerated lakelet system. Solids removed in the clarifier are mixed with the sanitary flow being sent to the regional plant.

The resuspension system in the SRC uses air and water injected through headers mounted on the walls of the tank. The solids slurry is removed through a sloped central channel in each tank and pumped across the river.

Solids collected in the retention basin are removed mechanically by front-end loaders after the basin has been dewatered. Final cleanup operations are aided by fire hoses mounted at intervals along the retention basin walls. An access ramp is provided to allow equipment into the basin.

Personnel Requirements--Based on an expected wet-weather operation of about 150 to 180 days per year, labor requirements of about 7.5 man-years will be required to operate the wet-weather facilities annually. The lakelet portion of the facilities will be operated year-round to maintain a consistent water quality so that recreational water quality standards are met. Approximately 35% of the labor hours will be required for actual facility operation, while the largest expenditure will be for maintenance, 48% [7, 8].

Alternate System Operation--

Until the dry-weather treatment facilities are modified for wet-weather use, operational steps for processing the combined flows from the retention basin include (1) pumping the resuspended solids and grit from the SRC to the dry-weather treatment facilities and mixing the solids with the city's sanitary flow for treatment and disposal, and (2) pumping the retention basin flows directly to the aerated lakelet system for biological treatment followed by sand filtration before discharge to the river.

Operational Problems--

A number of operational problems were identified during the operation of the aerated lakelet system in the demonstration project [3]:

- Icing of the surface aerators in the lakelets causing instability and possible capsizing.
- Algae growth, turbidity, and floating oil in the lakelets.
- Buildup of sludge deposits in the first lakelet.

Design changes in the treatment facilities provided in the citywide program are expected to reduce or eliminate these problems.

Winter conditions created significant ice buildup on surface aerators, causing instability and possible capsizing. This problem was corrected in the lakelets by replacing the surface aerators with a subsurface aeration system. Another possible solution to prevent icing would be to install heaters on the surface aerators.

Excessive quantities of algae were encountered in the operation of the lakelet portion of the demonstration project. In the recently constructed facilities, phosphorus removal in the treatment process can be provided if required to limit the growth of algae.

Turbidity in the lakelet system was encountered during the demonstration project, but lining the lakelets with stone should prevent the problem.

Oil in the combined sewage presented a problem during the operation of the lakelet system during the demonstration project; noticeable amounts of oil were present in the lakelets. The new facility, as designed, has a clarifier with a scum baffle, which should reduce the problem.

Sludge deposits were also a problem in the demonstration project with 23 cm (9 in.) or an estimated 800 $m^3$ (28,000 $ft^3$) occurring in the first lakelet. The large deposits were not anticipated in the demonstration project, indicating the problem of sampling and identifying the heavy solids occurring in combined sewage overflows. Taking representative samples is the key to accurate solids characterization. The use of the sedimentation-retention system and clarifiers at the modified dry-weather plant in the citywide facilities should reduce this problem. Odor problems in the retention basin should be minimized by the flushing system and the solids removal program.

## Performance

The total project implementation in Mount Clemens should reduce the volume of combined sewer overflows and pollutant loading to the Clinton River. The facilities are expected to meet effluent water quality criteria of 10 mg/L $BOD_5$ and 15 mg/L suspended solids, and emergency bypasses from the wet-weather treatment facilities that are projected to occur about 4 times a year would be disinfected. In addition to the performance of the combined sewage treatment processes, the effectiveness of the total system (including the impact of the sewer separation program) is evaluated in terms of load reduction to the river.

### Overflow Volume and Pollutant Reductions--

All of the volume of raw combined sewage will be eliminated from the Clinton River by full project implementation. The construction of the storage/transport tunnel and associated sewers to intercept the combined sewer overflow outfalls will collect the flows from 19 out of 24 overflow points [1, 7]. However, sewer separation will result in the discharge of stormwater at the 5 remaining overflow points.

About 95% of the suspended solids and $BOD_5$ in the combined sewage are projected to be removed by the wet-weather treatment facility. These projections, shown in Figure 111, are developed from the demonstration project data and the estimated treatment efficiencies for the physical processes (at conservative loading rates and operating conditions). The projected efficiencies of the treatment facility, either using the modified dry-weather plant or pumping directly to the lakelets, are about equal.

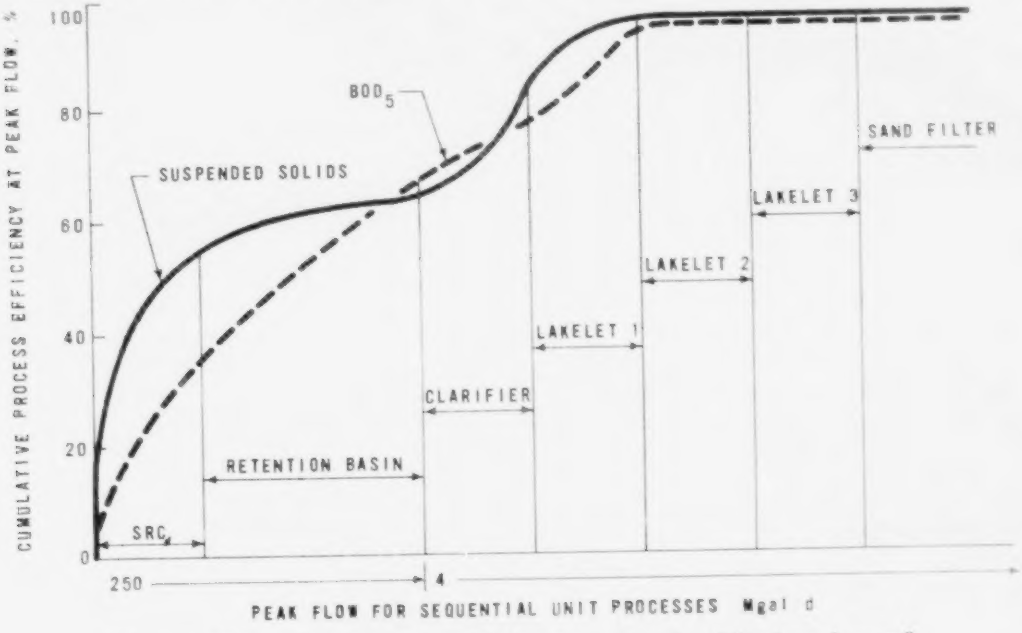


Figure 111. Projected pollutant removal efficiencies of Mount Clemens combined sewage treatment facility.

System Efficiency--

The projected pollutant loadings before and after full project implementation are shown in Figure 112. Annual loads were estimated for both combined sewage and stormwater runoff from the separated areas.

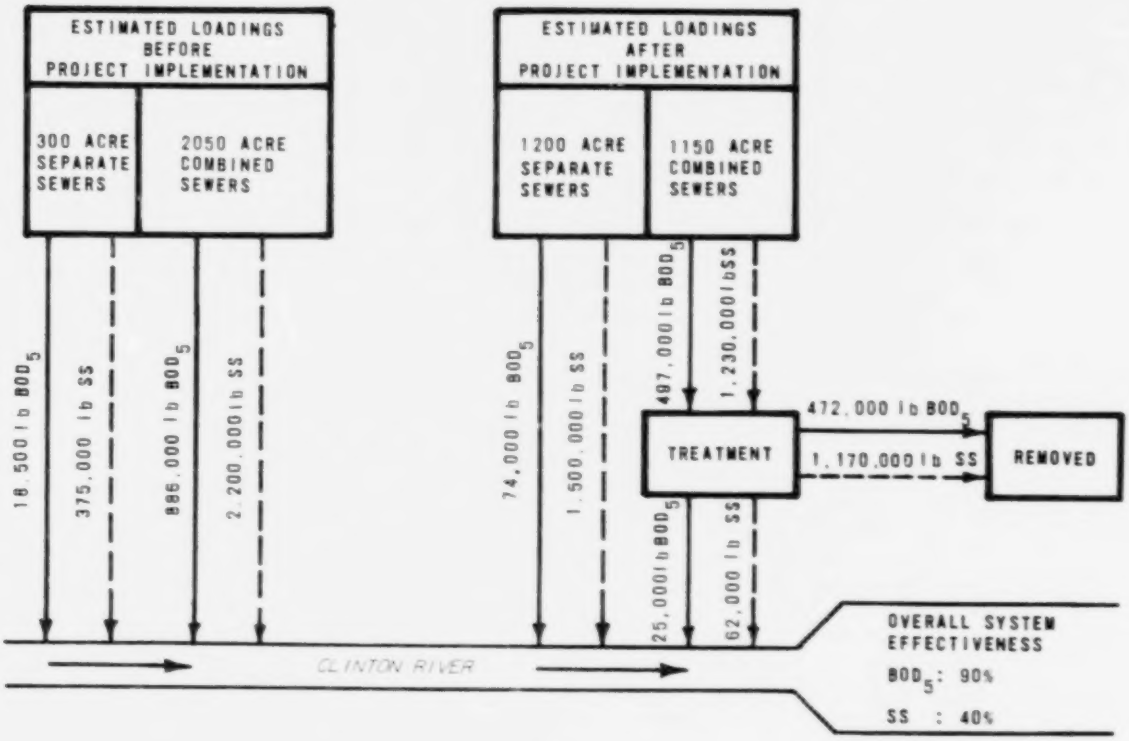


Figure 112. Comparison of estimated pollutant loads to the Clinton River before and after project implementation.

The stormwater runoff loads were developed assuming the pollutant concentrations of raw stormwater in Mount Clemens are comparable to the representative values of 20 mg/L for $BOD_5$ and 415 mg/L for suspended solids [9]. The assumptions also include the projected wet-weather facility treatment efficiency of 95% and do not assess the effect of emergency bypasses from the treatment facility.

A comparison of the annual load reduction achieved by project implementation with the preproject load estimates yields about a 90% $BOD_5$ and a 40% suspended solids reduction. Even though the combined sewage treatment efficiency is about 95% for both constituents, the stormwater loads from the separated areas are assumed to enter the river untreated. If suspended solids or associated heavy metals are a major criterion for maintaining the water quality of the receiving water, then the additional solids loading of stormwater discharges should be considered when separating sewers. The impacts of BOD loads in stormwater appear less significant than the solids load; however, oxygen demand may still be affected by other pollutants such as COD, or other organics.

Using just the loading values after project implementation from the separate and combined areas, the net annual load reduction is estimated at about 83% for $BOD_5$ and 43% for suspended solids.

## COSTS

Collection, storage, and treatment of combined sewage by physical and biological processes can be a most promising, cost-effective alternative for combined sewer overflow control where high degrees of pollutant removals are required.

### Capital Costs

The total cost to eliminate and treat combined sewer overflows was about \$21.5 million for a service area of 830 ha (2,050 acres), or about \$26,000/ha (\$10,500/acre). Estimated treatment capital costs, based on pollutant removal, are about \$38/kg (\$17/lb) $BOD_5$ and \$15/kg (\$7/lb) suspended solids or about \$0.005/L (\$0.02/gal) treated.[5]

Capital costs of the components of the Mount Clemens project included: (1) sewer separation, (2) construction of storage/transport system, and (3) construction of the combined sewer overflow treatment facilities (retention basin site and park treatment site). The cost of the park treatment site includes minor modification of the existing sanitary sewage treatment plant to treat combined sewage and development of the park system around the lakelets.

The capital costs of the system components and unit costs are summarized in Table 111.

Table 111. CAPITAL COSTS AND UNIT COSTS OF THE COMBINED SEWER OVERFLOW FACILITIES, MOUNT CLEMENS[a]

| Control measure | Total capital costs, \$ | Serviced area, acres | Annual combined overflow volume reduction, Mgal | Cost, \$/acre | Overflow volume reduction cost, \$/Mgal | Treatment cost, \$/gal |
|---|---|---|---|---|---|---|
| Sewer separation | 4,019,000 | 900 | 333[b] | 4,466 | 12,070 | -- |
| Interception | 8,916,000 | 1,150 | 426 | 7,753 | 21,000 | -- |
| Treatment | -- | 1,150 | 426 | -- | -- | -- |
| Retention basin site | 6,934,000 | -- | -- | 6,030 | -- | .016 |
| Park treatment site | 1,653,000 | -- | -- | 1,437 | -- | .002 |

a. ENR 3000.

b. Still allows stormwater volumes and pollutants to be discharged.

The Mount Clemens separation program was estimated to cost about \$1,800/ha (\$4,500/acre); about 42% less than the estimated regional average cost for the midwest of \$4,300/ha (\$10,700/acre) [10]. The net area separated in Mount Clemens, about 360 ha (900 acres), was achieved with a limited amount of new sewer construction because of the configuration and locations of the existing sewers. The areas that were separated were also of lower land use intensity than in the major cities for which separate costs were evaluated.

The capital costs of the retention basin and the storage/transport tunnel expressed as unit storage costs are \$68/m$^3$ (\$0.26/gal) for the retention basin and \$730/m$^3$ (\$2.75/gal) for the tunnel.

### Annual Costs

Annual operation and maintenance cost estimates of \$0.17/m$^3$ ((\$650/Mgal) of combined sewage treated are based on year-round operation of the lakelet system, operation of the retention basin when storm activated, and maintenance of the recreational area. The energy requirements for the treatment facilities are minimal in comparison to the total operation and maintenance budget. The operation and maintenance cost components are presented in Table 112.

Table 112. ESTIMATED OPERATION AND MAINTENANCE COSTS
MOUNT CLEMENS[a] [1, 3]

| Cost components | Estimated cost, \$/yr |
|---|---|
| Personnel | 165,000 |
| Supplies and equipment[b] | 40,000 |
| Chemicals | 10,000 |
| Power | 31,000 |
| Sludge disposal[c] | 18,000 |
| Other | 12,000 |
| Total | 276,000 |

a. ENR 3000.

b. Includes parts, materials, maintenance of vehicles, etc.

c. Sludge disposal to Detroit regional treatment facility.

## IMPACTS

The environmental and socioeconomic impacts of this project include the potential pollutant load reduction to the Clinton River and the benefits the public may derive from the multiuse park-treatment facility.

## Environmental Impacts

The reduction of pollutant loadings, through elimination of combined sewer overflows, is a necessary step in the overall revitalization of the Clinton River. The combined sewage treatment facility will remove an estimated 215,000 kg (472,00 lb) of $BOD_5$ and 531,000 kg (1,170,000 lb) of suspended solids annually from the river.

However, the estimated suspended solids discharged to the river from the separated areas is about 1.2 times the estimated solids load being treated. Although sewer separation effectively eliminates combined sewer overflows, the net receiving water benefit from sewer separation, in terms of solids loads, may be questionable. The potential adverse impacts of BOD from the separated area is substantially less than in combined sewer overflows. If the principal receiving water concern is dissolved oxygen depletion, then partial sewer separation in combination with the treatment of combined sewage may produce an environmental benefit.

The background receiving water quality in the Clinton River, before the citywide control facilities were constructed, was monitored for about 2-1/2 years [3]. BOD impacts to the river from all sources are considered minor. The dissolved oxygen level averaged about 7 mg/L. During the 2-1/2 years of monitoring, only about 8 out of the 79 days dissolved oxygen samples had values below 4 mg/L, or about 10% of the time. Assuming the sampling is representative, dissolved oxygen levels of below 4 mg/L would have occurred about 37 d/yr. Implementation of the combined sewage control program, including partial sewer separation, with the resulting high BOD load removal, should substantially reduce the number of days approaching dissolved oxygen violation limits.

The suspended solids concentration in the river averaged about 30 mg/L and ranged from less than 5 to about 200 mg/L. BOD averaged about 9 mg/L and ranged from 1 to 50 mg/L. Influences from storms can be seen in the data; however, the overall quality of the river is high. Combined sewage treatment as a part of the water pollution abatement program instituted by the state will hopefully return the river to an environment that will support game fish.

## Socioeconomic Impacts

Land use has been affected by the creation of the multiuse treatment facility. Even though Mount Clemens is nearly 100% developed, a portion of land was still unused prior to project implementation. The site of the park-treatment facility was a city landfill, bordered by land considered undesirable for development because of its proximity to the landfill.

The conversion of the landfill into a multiuse treatment facility, serving the public as a park-recreation area with a lake system, is a positive land use change, even though it involved a small percentage of the total city area. This change also made the surrounding undeveloped area more desirable property.

Although not completely landscaped, the lake system and recent development adjacent to the park site are shown in Figure 113, along with an artist's concept of the completed project.

High public acceptance of the project is due in part to the environmental awareness of the general public, but more as a result of providing a badly needed park-recreation area with small lakes for water sport. The elimination of the landfill as an undesirable area and improved aesthetics have increased the development value of the surrounding land.

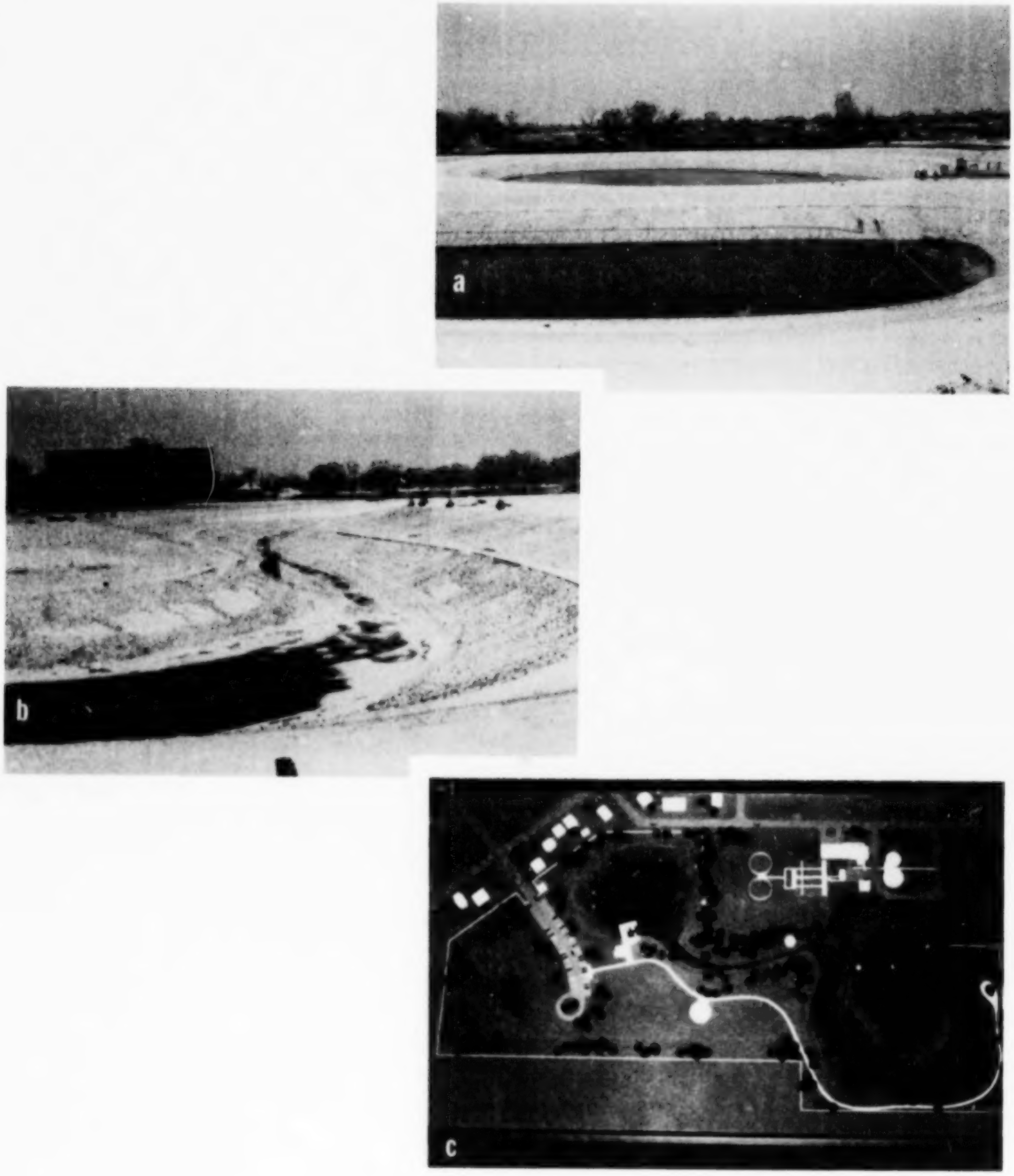


Figure 113. Present state of the Mount Clemens project: (a) a portion of the lakelet system, (b) lakelet and land adjacent to the project area, and (c) an artist's concept of completed park-treatment area.

# SECTION 15

## SWIRL REGULATOR/CONCENTRATOR DEMONSTRATION PROJECT
## LANCASTER, PENNSYLVANIA

A 7.3 m (24 ft) diameter dual-functioning swirl regulator/concentrator has been constructed in Lancaster, Pennsylvania, under a demonstration grant from the U.S. EPA Storm and Combined Sewer Section [1]. The swirl controls combined sewage overflows at one of the city's overflow points to the Conestoga River.

The original demonstration project was to include a storage silo as a part of the facilities, but the silo concept was dropped in 1974 because of the high construction bid cost of the project. The swirl project was continued, however; it was felt that adequate control could be achieved by the swirl alone. The swirl concentrator and a 2.4 m (8 ft) diameter swirl degritter were constructed and placed in service in early 1979.

Limited performance data have been collected on the swirl facilities because of sampling and flow monitoring difficulties. Estimates of performance, based on several storms that were monitored, indicate suspended solids mass removals ranging from 17 to 80%, depending on the size of the storm. This range includes the removals obtained from the hydraulic flow split in the swirl.

The swirl concentrator is one of the least expensive solutions to combined sewage overflow control. The cost of the Lancaster swirl is estimated at \$148,000/$\mathrm{m}^3$ s (\$6,500/Mgal d) of design capacity, or approximately \$3,360/ha (\$1,340/acre) of combined sewer service area. The representative construction cost for the swirl concentrator and appurtenant facilities at Lancaster was approximately \$13,900/ha (\$5,500/acre) of combined sewer service area.

The only other full-scale prototype swirl application on combined sewage in the United States is in Syracuse, New York. Mass suspended solids removal for this facility averaged 52% for 11 storms sampled [2, 3, 4].

### PROJECT DESCRIPTION

The demonstration project was developed to evaluate the feasibility of the swirl regulator/concentrator as a low cost, nonmechanical combined sewer overflow control device. The City of Lancaster is evaluating its applicability as a potential alternative to control combined sewer overflows to the Conestoga River. The principal features of the swirl are its compact space requirements, ability to operate as an overflow regulator and solids separator simultaneously, capability to operate at variable inflow rates, and low operation and maintenance requirements. Hydraulic modeling studies conducted during the development of the prototype design criteria indicate the swirl concentrator may remove over 90% of heavy solids (0.2 mm diameter at specific gravities of 2.65) [5].

## Area Characteristics

Lancaster is on the Conestoga River, a tributary of the Susquehanna River, in southeastern Pennsylvania, as shown in Figure 114. The area has gently rolling hills varying in elevation from 30 to 150 m (100 to 500 ft), and has well-drained soils. The city is about 100 km (60 mi) west of Philadelphia.

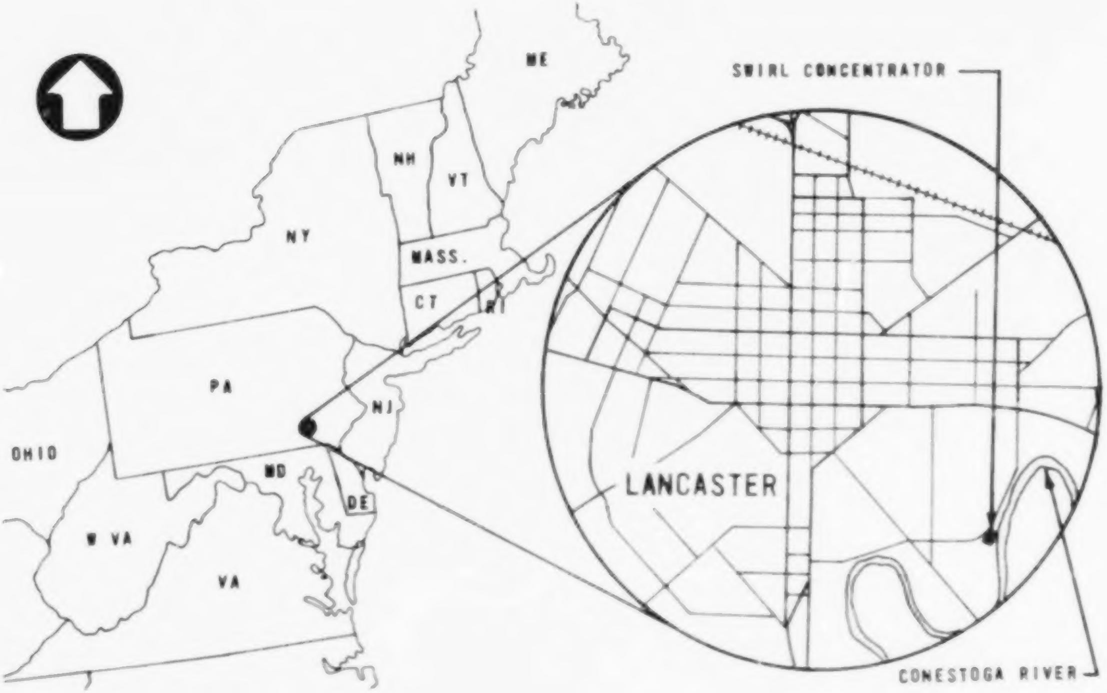


Figure 114. Lancaster, Pennsylvania.

The average annual precipitation is 110 cm (43 in.), which is about evenly distributed throughout the year. Slightly more precipitation occurs during the summer months (May through August) from thunderstorms. About 30 to 35 thunderstorms occur per year.

## Swirl Demonstration Project

Six combined sewage overflow points in Lancaster discharge untreated overflows to the Conestoga River during storms. During storms, the combined sewage discharge rates from these overflow points are estimated to range from 22,700 to 94,600 $m^3$/d (6 to 25 Mgal/d). These untreated flows can add substantial pollutant loads to the river. The river has a relatively high water quality during nonstorm periods. During the summer, BOD as low as 2 mg/L and total suspended solids as low as 70 mg/L have been recorded [6].

The swirl demonstration project was constructed at one of these overflow points to evaluate its effectiveness in controlling stormwater loads to the river.

Demonstration Area--

The 100 ha (250 acre) demonstration area is adjacent to the Conestoga River in the southeastern portion of the city, as shown in Figure 115. The developed portion of the area, about 50 ha (125 acres), is residential. The remaining area is parklands or undeveloped fields.

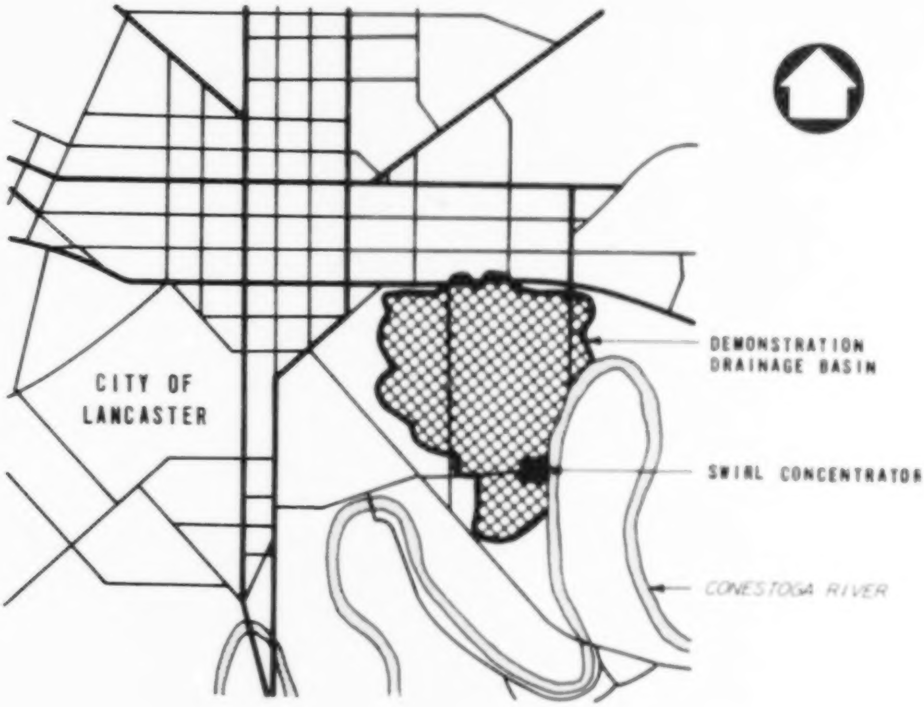


Figure 115. Swirl concentrator demonstration drainage basin in Lancaster [7].

The developed area has an estimated runoff coefficient of 0.59. The undeveloped half of the drainage area is assumed to contribute runoff directly to the river [7].

### Swirl Facilities--

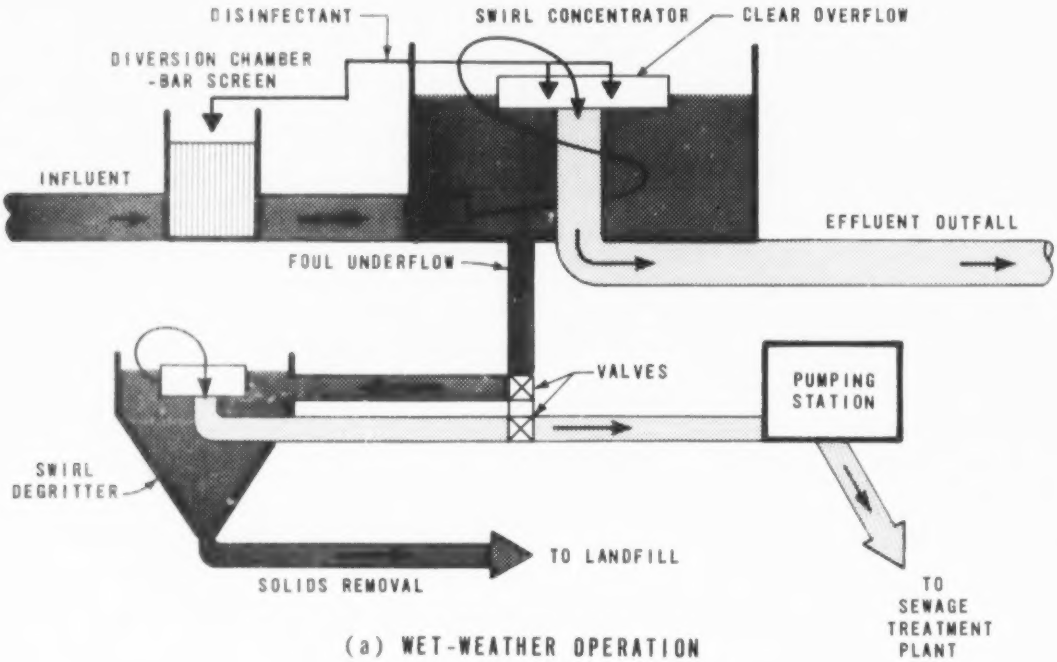
A 7.3 m (24 ft) diameter swirl concentrator was constructed to replace an existing combined sewer overflow regulator on a 1.5 m (60 in.) reinforced-concrete sewer. The swirl was designed to operate at 1.1 $m^3/s$ (40 $ft^3/s$) with the capacity to be surcharged to 2.6 $m^3/s$ (90 $ft^3/s$). An emergency bypass was constructed in the wall of the swirl to pass flows up to 3.5 $m^3/s$ (125 $ft^3/s$) to the river without flooding the swirl unit.

A swirl degritter and grit handling system was constructed to remove grit from the foul underflow of the swirl concentrator before the underflow is pumped to the municipal sewage treatment plant. The grit is conveyed to a storage bin and disposed of at a landfill. Disinfection of the swirl concentrator clear overflow is provided. Disinfection of raw overflows can also be accomplished at the diversion chamber ahead of the swirl facilities. A schematic of the facilities during wet- and dry-weather operation is shown in Figure 116. During dry weather, sewage enters the swirl concentrator and is directed along the floor gutters to the foul underflow outlet and then directly to the pumping station.

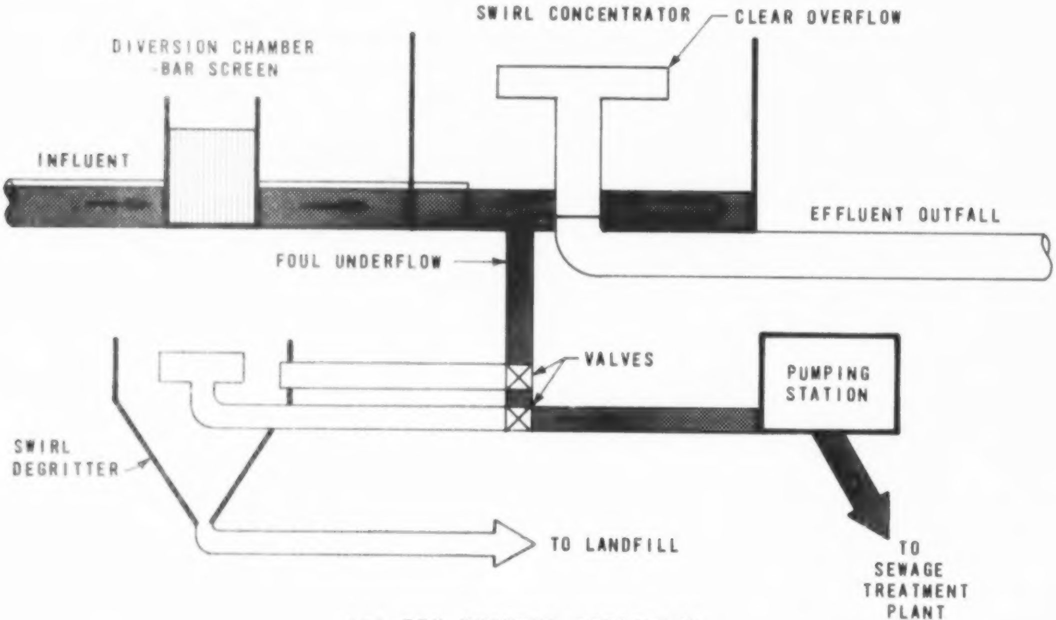
Flows from the combined sewer enter a diversion chamber and bar screen before entering the swirl. The bar screen protects downstream facility components from large pieces of debris. The diversion chamber has an emergency bypass to prevent the swirl facilities from being flooded. The bypass is activated by a sonic level sensor that opens a bypass gate when flows into the chamber exceed a preset elevation. All bypassed flows are chlorinated.

The swirl facilities are automatically activated when storm flows enter the swirl. A sonic level sensor activates a sequence of valves to change the mode from dry- to wet-weather operation. During wet-weather operation, the combined sewage is split into a clear overflow stream, which is chlorinated and discharged to the river and the foul underflow stream. The swirl facilities are shown in Figure 117.

The foul underflow to the swirl degritter was initially controlled by a pinch valve linked to a magnetic flowmeter. Because of the slow response time between the flowmeter and the control valve, flows above the hydraulic design capacity of the swirl degritter could occur instantaneously as head built up in the swirl concentrator. A Hydrobrake was installed in the foul underflow line to correct this problem. The Hydrobrake, with a 17.8 cm (7 in.) opening at its constriction, is a static device that controls flow by increasing headloss through the unit. The headloss is created by vortex fluid motion. As the head builds up above the Hydrobrake, the fluid rotation increases, thereby increasing the headloss through the orifice and decreasing the flow. The unit was designed to pass a maximum flow of 0.06 $m^3/s$ (2 $ft^3/s$). The anticipated performance of the Hydrobrake compared with a conventional orifice is shown in Figure 118. A schematic of the Hydrobrake operation is shown in Figure 119.



(a) WET-WEATHER OPERATION



(b) DRY-WEATHER OPERATION

Figure 116. Schematic of the Lancaster swirl regulator/concentrator facilities.

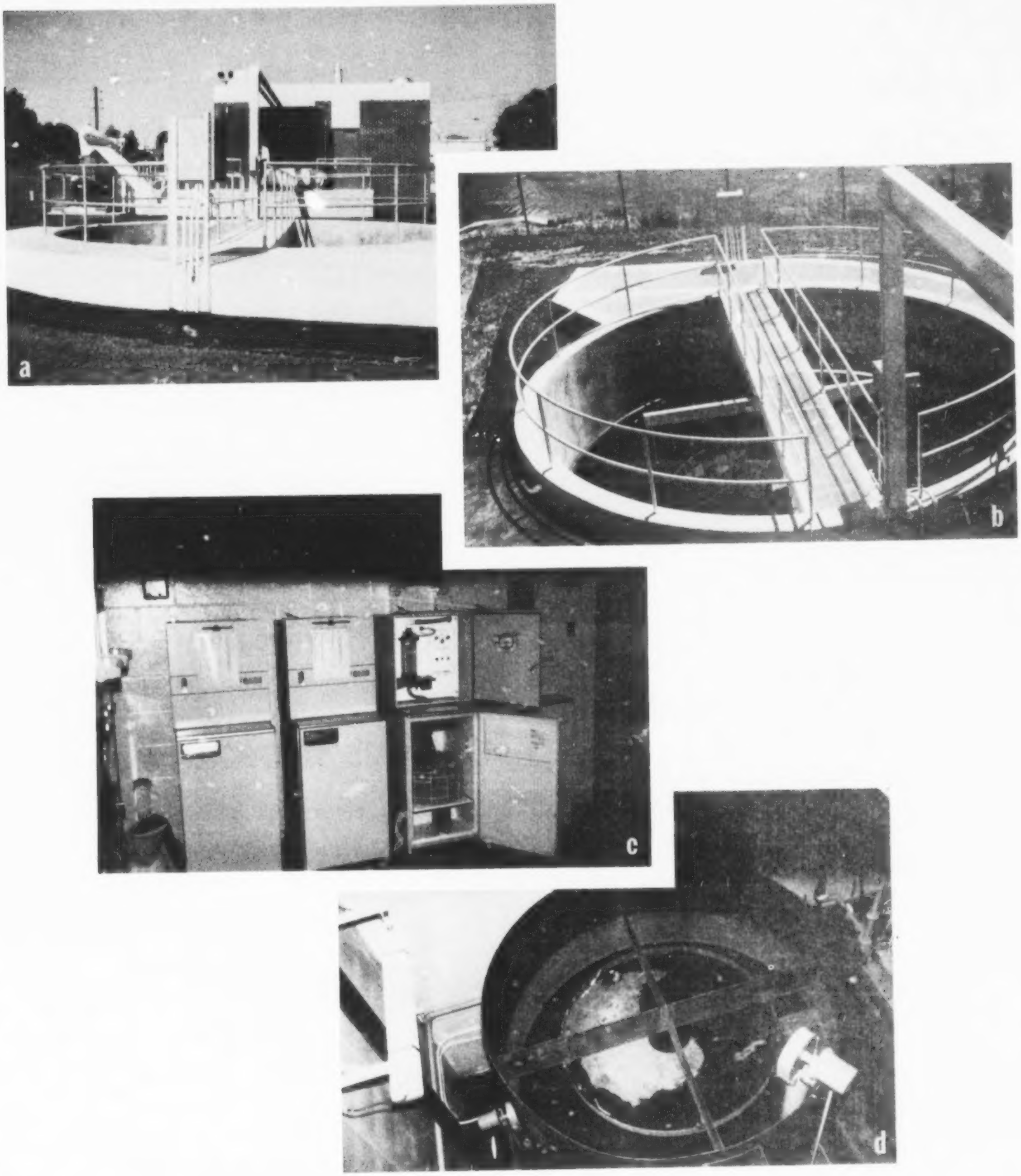


Figure 117. Lancaster swirl facilities: (a) 24 ft swirl regulator/concentrator and control building, (b) swirl regulator/concentrator, (c) automatic samplers, and (d) 8 ft swirl degritter.

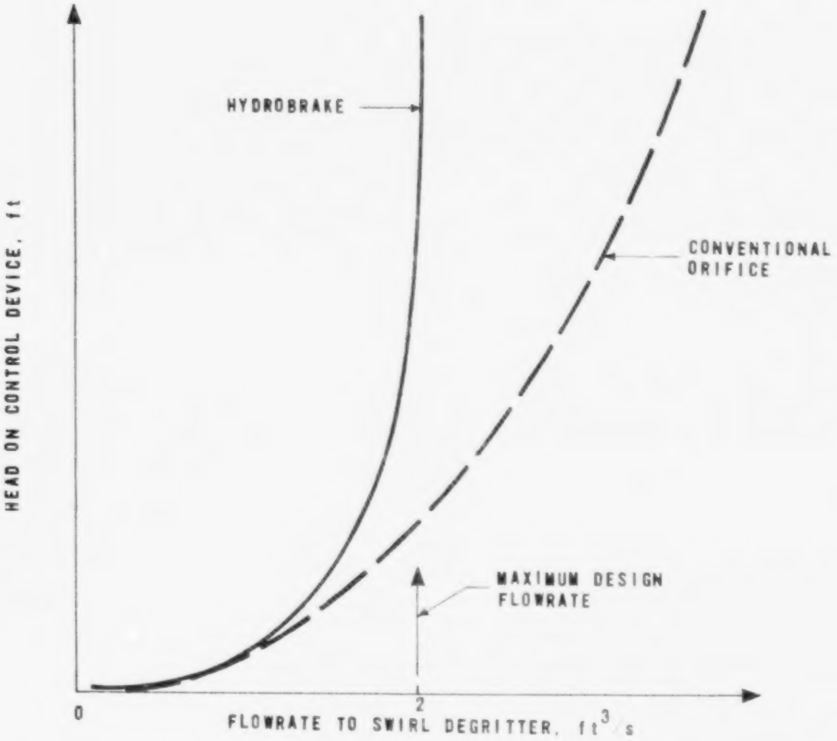


Figure 118. Conceptual performance comparison of the Hydrobrake and a conventional orifice.

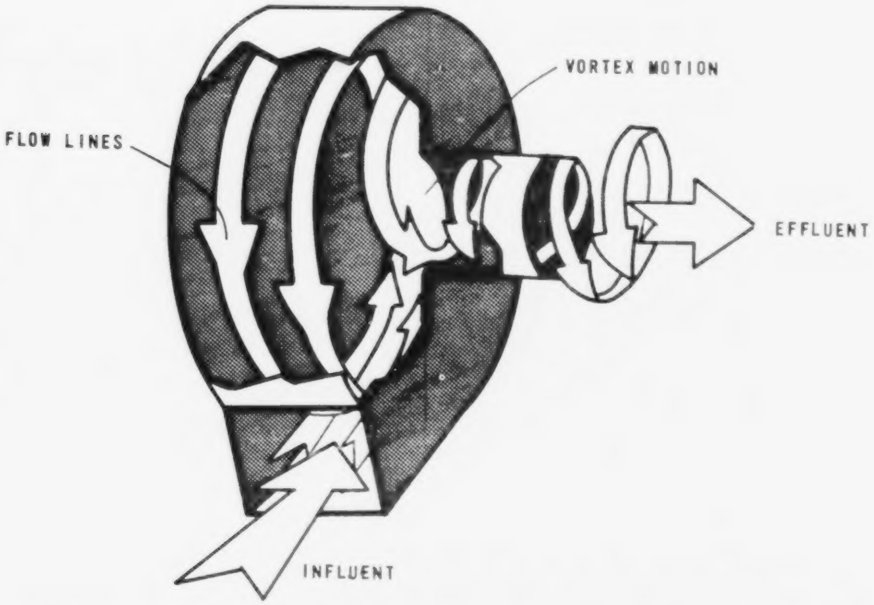


Figure 119. Schematic of Hydrobrake operation showing principal of vortex fluid motion.

The demonstration facilities also have a 9.5 L/s (150 gal/min) discostrainer that is being evaluated on the swirl foul underflow. The facilities' principal sampling system has four automatic samplers that can pull samples from 12 different locations in the system.

## Swirl Regulator/Concentrator Design

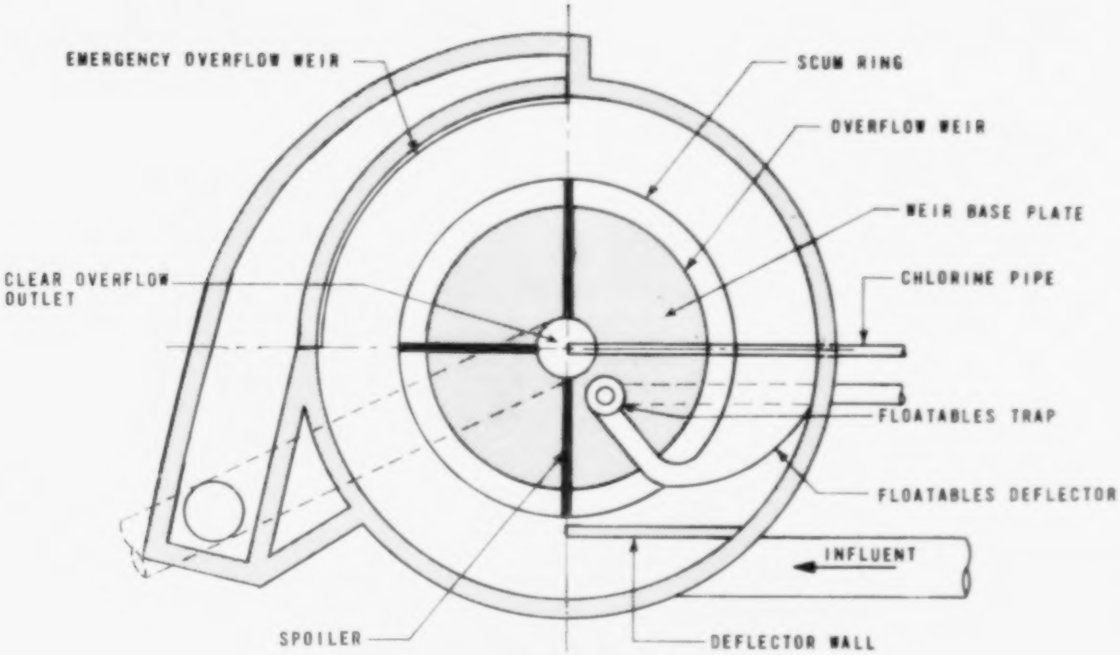
The swirl regulator/concentrator design criteria were developed through mathematical and hydraulic model simulation [1, 8]. Synthetic settleable solids, grit, and floatables were used in the hydraulic testing to simulate combined sewage solids. The specific characteristics of these solids are summarized in Table 113.

Table 113. CHARACTERISTICS OF THE SOLIDS USED IN DEVELOPING SWIRL DESIGN RELATIONSHIPS THROUGH MODEL SIMULATION [8]

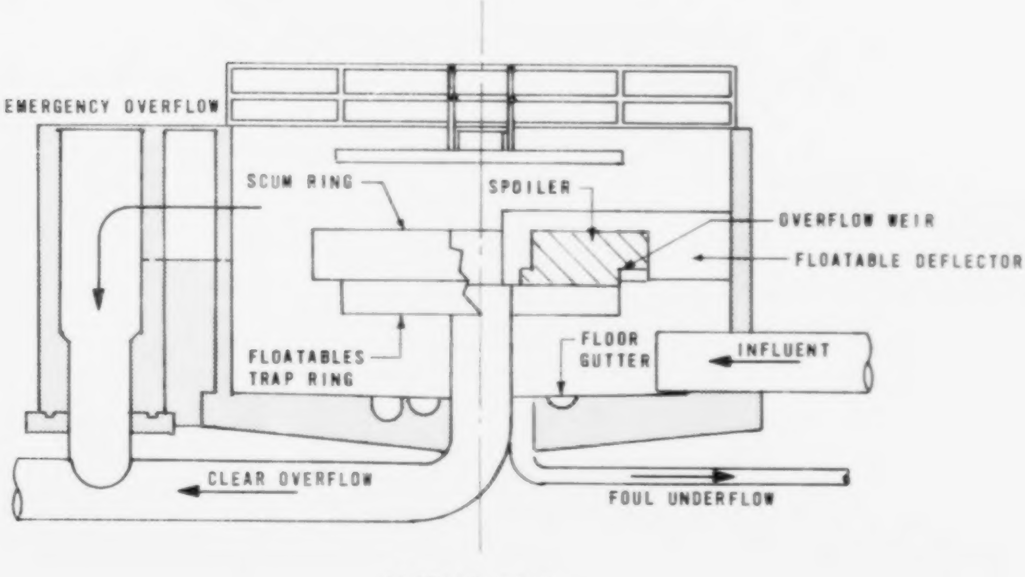
| Material | Specific gravity | Concentration, mg/L | Particle distribution size, mm | Particle distribution % by weight |
|---|---|---|---|---|
| Settleable solids (excluding grit) | 1.05-1.2 | 200-1,550 | 0.2 | 10 |
| | | | 0.5 | 10 |
| | | | 1.0 | 15 |
| | | | 2.5 | 25 |
| | | | 5.0 | 40 |
| Grit | 2.65 | 20-360 | 0.2 | 10 |
| | | | 0.5 | 10 |
| | | | 1.0 | 15 |
| | | | 1.5 | 25 |
| | | | 2.0 | 40 |
| Floatable solids | 0.9-0.998 | 10-80 | 5 | 10 |
| | | | 10 | 10 |
| | | | 15 | 20 |
| | | | 20 | 20 |
| | | | 25 | 40 |

The model was tested at flows simulating the range from 0.4 to 4.7 $m^3/s$ (15 to 165 $ft^3/s$), which were representative of the flows expected at the Lancaster facility [8, 9].

The design relationships developed in the modeling studies were used to design the 7.3 m (24 ft) diameter swirl concentrator [5]. The depth of the swirl, base to weir crest, is 1.8 m (6 ft). A plan and section view of the swirl are shown in Figure 120.



TOP VIEW



SECTION VIEW

Figure 120. Details of the Lancaster swirl regulator/concentrator.

The swirl is operated as a gravity flow system that acts as a flow regulator, provides pollutant removal, and has no moving parts. The major components of the swirl include the:

- Flow deflector and spoilers
- Clear overflow and foul underflow outlets
- Scum ring and floatables collector and floor gutters

### Flow Deflector and Spoilers--

The flow deflector and spoilers maintain the desired flow regime in the swirl tank--circular flow patterns that set up the secondary fluid motion. The flow deflector helps prevent mixing or flow disturbance between the influent flow and the volume already in the circular flow pattern. It also deflects the flow toward the center of the swirl, increasing the swirl velocity.

The spoilers, which are a part of the clear overflow weir, prevent the formation of a free surface vortex at the outlet by creating perpendicular resistance to the circular flows at the surface. The spoilers also create a relatively quiescent area near the overflow weir so that a uniform flow over the weir is achieved.

### Clear Overflow and Foul Underflow Outlets--

The clear overflow and the foul underflow outlets control the flow split in the swirl. The Lancaster swirl has a 4 m (13.3 ft) diameter clear overflow weir ring with 13 m (42 ft) of weir. The clear overflow is discharged through the effluent outfall to the river through a 0.9 m (3 ft) diameter drop shaft in the center of the circular weir baseplate.

The foul underflow outlet is a 0.3 m (1 ft) diameter pipe connected to the base of the swirl. During dry weather, municipal sewage is routed through the swirl to the foul outlet and on to the sewage treatment plant. During wet-weather operation, approximately 5% of the influent flow, containing the concentrated solids, is removed through the foul outlet and sent to the swirl degritter.

### Scum Ring, Floatables Collector, and Floor Gutters--

The scum ring and the floatables collector are attached to the clear overflow weir assembly and prevent floatables from overflowing with the clear effluent. The scum ring acts as a baffle, keeping the floatables outside the clear overflow weir.

The floatables are then directed by the floatable deflector, a barrier across the surface of the water, to a floatables trap. The trap is connected to a 5.7 $m^3$ (200 $ft^3$) floatable storage area under the clear overflow weir plate. The storage area is formed by a trap ring that is an extension of the clear overflow weir, 0.5 m (1.5 ft) below the weir plate. After a storm, the trapped floatables enter the foul underflow outlet as the water level falls and are transported to the sewage treatment plant.

The floor gutters form a contracting spiral from the swirl influent entrance on the periphery of the concentrator base to the foul outlet near the center of the concentrator. The gutters direct dry-weather flow through the swirl during nonstorm periods and guide the concentrated combined solids to the center of the swirl during wet-weather operation.

#### Other Features--

The Lancaster swirl also has disinfection facilities, a flushing system, and an emergency overflow weir. All of the clear overflow effluent is chlorinated before discharge to the river. A chlorine pipe extends over the clear overflow weir assembly and doses the effluent just before it is discharged.

Flushing lines are located on the sidewalls above the waterline and under the floatables trap area to wash deposits from the swirl after each storm. The deposits are washed into the floor gutters and are sent to the treatment plant.

The emergency overflow weir covers one-quarter of the sidewall of the swirl. The weir is 15.2 cm (6 in.) higher than the clear overflow weir and provides an additional 6 m (19 ft) of weir length for flows that exceed the hydraulic capacity of the swirl. Details of the swirl concentrator components are shown in Figure 121.

## OPERATION AND PERFORMANCE

During the project evaluation phase of the demonstration project, several operational problems were encountered that resulted in a limited amount of useful data being collected. Most of the problems were with the monitoring and sample collection. These problems are discussed in the following and offer guidance for future design of monitoring systems.

### Operational Problems

The major problems during the evaluation of the swirl were:

- Malfunctioning flowmeters
- Sampling problems
- Clear overflow weir level fluctuations

#### Flowmeters--

Malfunctioning flowmeters have been a continuous problem during the first year of operation. Two types of flowmeters were used in the facilities: a sonic flowmeter to measure the swirl influent flow and a magnetic flowmeter coupled with a control valve to throttle the foul underflow to the swirl degritter.

The sonic flowmeter in the 0.9 m (3 ft) swirl influent line would "zero out" whenever the pipe was surcharged. Under this condition, measurement of influent pollutant loads is reduced to guesswork. This significantly affects the facility performance evaluation, i.e., the accuracy of the mass balance is

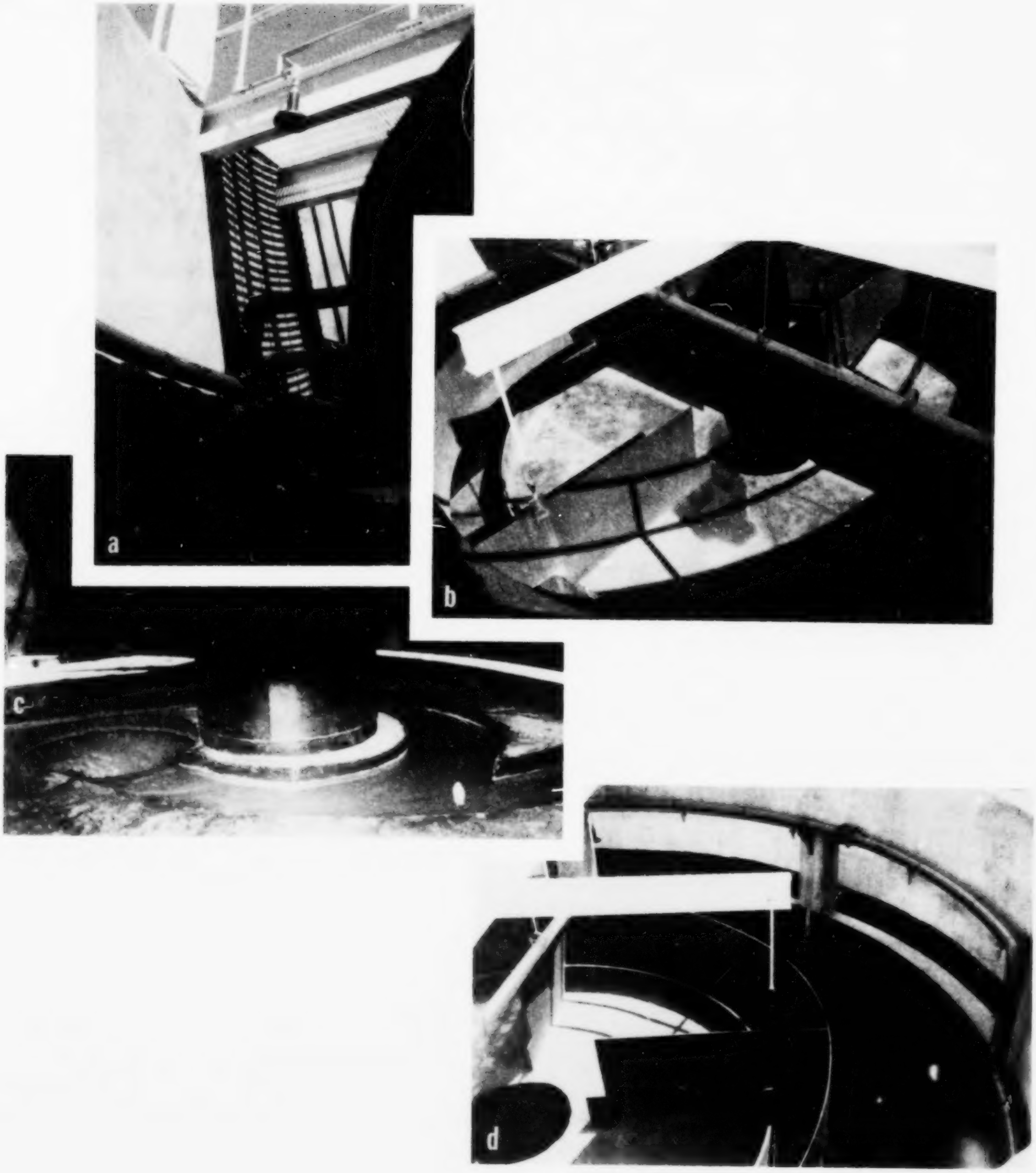


Figure 121. Lancaster swirl concentrator components: (a) sonic level sensor suspended from catwalk and flushing waterlines: (b) clear overflow weir assembly showing spoilers, floatables trap, scum baffle, and chlorine line; (c) floor gutters and foul underflow outlet; and (d) clear overflow weir and emergency overflow weir and orifice along sidewall.

constrained by the accuracy of the flow measurement. Physical methods of flow measurement, such as depth of flow over a weir or other control surface, may result in more reliable data, or at least provide a check on the passive flow measurement devices.

The problem of exceeding the design flow capacity of the swirl degritter caused by the response delay between the magnetic flowmeter and the control valve was corrected by the installation of a Hydrobrake. A limit switch was placed on the control valve to prevent wide-open operation.

Automatic Samplers--

Automatic sampler problems have been identified as the probable cause of inconsistent and unrepresentative settleable solids characterization. The swirl is basically a treatment device for the heavier solids fraction and grit and conventional sampling practices are not suited for the accurate characterization of these solids without several design considerations to account for better representation of heavier solids. These considerations include multiple sampler intake locations, higher sampler intake velocities, shorter sampler pipe runs, and higher sampler pipe velocities.

Several sample piping runs are over 15 m (50 ft) long and have numerous fittings and changes of direction. For example, the samples from the influent combined flow pass through a least seven ells, two valves, five unions, and three tees before reaching the sample pump. The discharge from the pump goes through similar fittings, including a 6 m (20 ft) horizontal run and a long vertical run before reaching the sampler. Sediment can build up in horizontal pipe runs and, if velocities are low, the vertical pipe run could act as a tube settler. The sampling system flow velocities range from 0.82 to 1.04 m/s (2.7 to 3.4 ft/s) in the sampling loop.

In general, samplers should be located as close to the point being sampled as practical, to reduce long piping runs. Sample intake locations should consider the occurrence of stratified solids flow in the stream being sampled. Sample velocities should be uniform throughout the sampling system and should at least match the velocity of the stream being sampled, or be at least 1.2 m/s (4 ft/s). These considerations are viewed as critical in obtaining representative samples of the heavy solids fraction.

Clear Overflow Weir--

The clear overflow weir in the swirl has experienced level changes that can result in an uneven flow distribution over the weir. The problem is assumed to be caused by the heating and cooling of the suspension rods holding up the weir. Protection of the swirl by a sunshade has been suggested.

## Performance

Data on the performance of the Lancaster swirl regulator/concentrator are limited: only 4 storms of the 10 storms monitored have sufficient flow and quality data to permit mass removal evaluations. The sampling data collected for settleable solids may not be representative because of the low sampling velocities and long piping runs to the samplers. Little BOD data were collected. Therefore, suspended solids was the only parameter evaluated. The potential swirl effectiveness, however, may be inferred from the results of the Syracuse, New York, swirl studies [2, 3], until the Lancaster facilities collect a statistically reliable data base.

Mass removal of suspended solids is achieved by two processes: (1) the flow split between the clear overflow and the foul overflow, and (2) the concentration effect of the swirl motion. The mass suspended solids removals for the four storms ranged from 17 to 80%, with hydraulic flow splits ranging from 15 to 48%, respectively. The size of the storms ranged from very small, about 10% of the design flow capacity, to a storm that equaled or exceeded the design flow capacity. A summary of the storm data and swirl effectiveness is presented in Table 114.

Table 114. SWIRL REGULATOR/CONCENTRATOR EFFECTIVENESS, LANCASTER

| | | Storm flows | | | | Suspended solids removal, %[a] | | |
|---|---|---|---|---|---|---|---|---|
| Storm date | Storm duration, h | Peak, ft$^3$/s | Ratio to design flow, % | Average, ft$^3$/s | Ratio to design flow, % | Mass removal | Concentration reduction | Hydraulic flow split, %[b] |
| 3/5/79 | 1.7 | 6.8 | 17 | 3.5 | 9 | 78 | 64 | 40 |
| 4/4/79 | 1.1 | 6.5 | 16 | 3.8 | 10 | 76 | 54 | 48 |
| 5/23/79 | 2.2 | >40[c] | >100 | >10.4 | >26 | 17 | 2 | 15 |
| 5/26/79 | 0.5 | 4.0 | 10 | 2.3 | 6 | 80 | 60 | 49 |

a. Flow-weighted averages.

b. Ratio of foul underflow volume to total influent volume. These values are also representative of the mass removal of suspended solids resulting from the flow split.

c. Estimated to be at least 40 ft$^3$/s; flowmeter reads zero during surcharged pipe flow condition.

The size of the storm inflow volume and the split between the foul underflow and the clear overflow significantly affect the overall performance of the unit. For example, a very small storm could have up to 50% of its total storm volume diverted through the foul outlet, thereby netting at least a 50% mass removal independent of any concentration effects. Since the foul underflow rate is controlled and roughly the same for all storms 0.3 to 0.6 m/s (1 to 2 ft$^3$/s), large storms would have a relatively small fraction of the total storm volume diverted through the foul outlet. The corresponding mass removal by the flow split is therefore much smaller.

Other considerations, such as timing of the influent load (first flush), the flowrate, and the storm duration, affect performance. For example, during the storm of May 23, 1979, 76% of the total mass suspended solids load reached the swirl in the first 14 minutes of the 2.2 hour runoff event. This corresponds with 50% of the total flow volume during the same time and the foul underflow volume was only 2% of the total inflow volume. Even if the concentration of the foul underflow were ten times the concentration of the influent flow, only about 16% of the total influent load during the 14 minute operation period would have been removed (76% x [2% x 10]).

Tests conducted on a 3.7 m (12 ft) diameter swirl concentrator in Syracuse, New York, indicated that the mass suspended solids reduction averaged 52% for 11 different storms. The removals ranged from about 30 to 80%. The estimated average mass removal obtained by the flow split was about 33%. Therefore, the average expected contribution from solids concentration could be about an additional 20% removal [2, 3].

## COSTS

The swirl regulator/concentrator is the least expensive structural alternative for controlling combined sewer overflows. Until a reliable and much larger performance data base is available, complete judgment on the cost effectiveness of the swirl is not possible. Therefore, none are presented in this text. Since the Lancaster project was a demonstration project, the actual demonstration capital costs and the operation and maintenance costs of the project may be higher than would be required for a normal prototype facility.

### Construction Cost

The total cost of the Lancaster swirl regulator/concentrator demonstration project was about $1,100,000 (ENR 3000). In addition to the construction cost of the swirl facilities, this cost includes inspection and administration, engineering, equipment, supplies, laboratory charges, operating salaries, utilities, repairs, and the project report.

The construction costs of the swirl facilities were estimated at about $692,000, of which the swirl regulator/concentrator cost was estimated at about $168,000 or about $148,000/m$^3$·s ($6,500/Mgal·d) of design capacity. The other construction costs include the grit swirl and grit handling system, the control building (which also houses the grit swirl), the instrumentation, and the disinfection system. The costs are summarized in Table 115.

Table 115. SUMMARY OF SWIRL FACILITIES CONSTRUCTION COSTS, LANCASTER[a]

| Item | Cost, $ |
|---|---|
| Swirl concentrator | 168,000 |
| Swirl degritter and grit handling system | 56,000 |
| Control building | 255,000 |
| Instrumentation | 146,000 |
| Disinfection system | 67,000 |
| Total | 692,000 |

[a] ENR 3000.

## Operation and Maintenance Costs

The operation and maintenance cost for the first year of operation of the demonstration facilities is estimated at about $58,000. The cost includes operating personnel salaries, supplies, utilities, and a budget for equipment repairs or replacement.

There is a large potential for startup problems and facilities debugging efforts with any newly constructed facility. The annual operation and maintenance budget after the startup period and the demonstration period is expected to be about one-half the first year cost, about $26,000.

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# GLOSSARY

Aerated lagoon--A natural or artificial wastewater treatment lagoon (generally from 4 to 12 feet deep) in which mechanical or diffused-air aeration is used to supplement the oxygen supply.

Benthic--Of or pertaining to the bottom of streams, rivers, lakes, or oceans.

Biological treatment processes--Means of treatment in which bacterial or biochemical action is intensified to stabilize, oxidize, and nitrify the unstable organic matter present. Trickling filters, activated sludge processes, and lagoons are examples.

BMP--Best Management Practices. Nonstructural and low structurally intensive measures for controlling stormwater pollution by correcting the problem at its source.

BOD--Biochemical Oxygen Demand. The quantity of dissolved oxygen used by microorganisms in the biochemical oxidation of organic matter and oxidizable inorganic matter by aerobic biological action. Generally refers to the standard 5-day BOD test.

Combined sewage--Sewage containing both domestic sewage and surface water or stormwater, with or without industrial wastes. Includes flow in heavily infiltrated sanitary sewer systems as well as combined sewer systems.

Combined sewer--A sewer receiving both intercepted surface runoff and municipal sewage.

Combined sewer overflow--Flow from a combined sewer in excess of the interceptor capacity that is discharged into a receiving water.

COD--Chemical Oxygen Demand. The quantity of oxygen required to oxidize organic matter in the presence of a strong oxidizing agent in an acidic medium.

CSO--Combined Sewer Overflow.

Curve numbers (CN)--A table of numbers developed by the Soil Conservation Service to calculate stormwater runoff for various land surface conditions. The numbers are a function of land use, hydrologic soil group, and antecedent moisture conditions.

Detention--The slowing, dampening, or attenuating of flows either entering the sewer system or within the sewer system by temporarily holding the water on a surface area, in a storage basin, or within the sewer itself.

Disinfection--The destruction of the larger portion of microorganisms in or on a substance with the probability that all pathogenic bacteria are killed by the agent used.

Diversity--A measure of the relationship between the number of species and the total population of a biological community. Values determined by the Shannon-Weaver formula.

Domestic sewage--Sewage derived principally from dwellings, business buildings, institutions, and the like. It may or may not contain groundwater.

Dry weather flow--Base flow in sanitary or combined sewers including sanitary, industrial, and inflow/infiltration flows during nonstorm periods.

Dual treatment--Those processes or facilities designed for operating on both dry- and wet-weather flows.

Dynamic regulator--A semiautomatic or automatic regulator device which may or may not have movable parts that are sensitive to hydraulic conditions at their points of installation and are capable of adjusting themselves to variations in such conditions or of being adjusted by remote control to meet hydraulic conditions at points of installation or at other points in the total combined sewer system.

Equalization--The averaging (or method for averaging) of variations in flow and composition of a liquid.

Eutrophication--The maturing natural or artificial process of a water body, charactorized by high concentrations of nutrients and periods of oxygen deficiency.

First flush--The condition, often occurring in storm sewer discharges and combined sewer overflows, in which a disproportionately high pollutional load is carried in the first portion of the discharge or overflow.

Infiltration--The water entering a sewer system and service connections from the ground, through such means as, but not limited to, defective pipes, pipe joints, connections, or manhole walls. Infiltration does not include, and is distinguished from, inflow.

Inflow--The water discharged into a sewer system and service connections from such sources as, but not limited to, roof leaders, cellar, yard, and area drains, foundation drains, cooling water discharges, drains from springs and swampy areas, manhole covers, cross connections from storm sewers and combined sewers, catchbasins, stormwaters, surface runoff, street wash waters, or drainage. Inflow does not include, and is distinguished from, infiltration.

In-system--Within the physical confines of the sewer pipe network or treatment system.

Intercepted surface runoff--That portion of surface runoff that enters a sewer, either storm or combined, directly through catchbasins, inlets, etc.

Interceptor (intercepting sewer)--A sewer that receives dry-weather or wet-weather flow from a number of transverse combined sewers and additional predetermined quantities of intercepted surface runoff and conveys such waters to a point for treatment.

Intermittent point sources--Any discernible, confined, and discrete conveyance from which pollutants are or may be discharged on a noncontinuous basis.

Municipal sewage--Sewage from a community. May be composed of domestic sewage, industrial wastes, or both.

Natural treatment--The use of wetlands, marshes, swamps, etc., for the reduction of pollutant concentrations of stormwater or secondary effluent from sanitary sewage treatment plants.

Nonpoint source--Any unconfined and nondiscrete conveyance from which pollutants are or may be discharged.

Nonsewered urban runoff--That part of the precipitation that runs off the surface of an urban drainage area and reaches a stream or other body of water without passing through a sewer system.

Offsite storage--Source detention methods used at a location that is downstream or separated from the site or sites being controlled. Offsite storage is used to control stormwater from developed areas.

Onsite storage--Source detention methods used directly on the site being controlled.

Overflow--(1) The flow discharging from a sewer resulting from combined sewage, stormwater, or extraneous flows and normal flows that exceed the sewer capacity. (2) The location at which such flows leave the sewer.

Physical-chemical treatment processes--Means of treatment in which the removal of pollutants is brought about primarily by chemical clarification in conjunction with physical processes. The full process string generally includes preliminary treatment, chemical clarification, filtering, carbon absorption, and disinfection.

Physical treatment operations--Means of treatment in which the application of physical forces predominates. Screening, sedimentation, flotation, and filtration are examples. Physical treatment operations may or may not include chemical additions.

Point source--Any discernible, confined, and discrete conveyance from which pollutants are or may be discharged.

Pollutant--Any harmful or objectionable material in or change in the physical characteristic of water or sewage.

Primary productivity--The rate at which energy-containing materials are formed by plants usually measured as mg carbon/$m^2 \cdot d$. The rate is a function of water quality, substrate, temperature (seasonal), and solar illumination.

Pretreatment--The removal of material such as gross solids, grit, grease, and scum from sewage flows prior to physical, biological, or physical-chemical treatment processes to improve treatability. Pretreatment may include screening, grit removal, skimming, preaeration, and flocculation.

Regulator--A structure that controls the amount of sewage entering an interceptor by storing in a trunk line or diverting some portion of the flow to an outfall.

Retention--The prevention of runoff from entering the drainage system or downstream water body by storing on a surface area or in a storage basin.

Sanitary sewer--A sewer that carries liquid and water-carried wastes from residences, commercial buildings, industrial plants, and institutions, together with relatively low quantities of ground, storm, and surface waters.

SCS--Soil Conservation Service.

Sewer--A pipe or conduit generally closed, but normally not flowing full, for carrying sewage, stormwater, or other waste liquids.

Sewerage--System of piping, with appurtenances, for collecting and conveying wastewaters from source to discharge.

SG--Specific gravity.

Static regulator--A regulator device that has no moving parts or has movable parts which are insensitive to hydraulic conditions at the point of installation and which are not capable of adjusting themselves to meet varying flow or level conditions in the regulator-overflow structure.

Storm flow--Overland flow, sewer flow, or receiving stream flow caused totally or partially by storm runoff or snowmelt.

Storm sewer--A sewer that carries intercepted surface runoff, street wash and other wash waters, or drainage; designed to exclude domestic sewage and industrial wastes.

Storm sewer discharge--Flow from a storm sewer that is discharged into a receiving water.

Stormwater--Water resulting from precipitation that either percolates into the soil, runs off freely from the surface, or is captured by storm sewer, combined sewer, and to a limited degree sanitary sewer facilities.

Surcharge--The flow condition occurring in closed conduits when the hydraulic grade line is above the crown of the sewer.

Surface runoff--Precipitation that falls onto the surface of roofs, streets, ground, etc., and is not absorbed or retained by that surface, thereby collecting and running off.

Trunk sewer--That portion of a sanitary, storm, or combined sewer that accepts flow from collectors and laterals for transport to an interceptor.

Urban runoff--Surface runoff from an urban drainage area that reaches a stream or other body of water or a sewer.

Wastewater--The spent water of a community. See Municipal Sewage and Combined Sewage.

Wet-weather flow--Dry-weather flow plus surface water, stormwater, and/or excess inflow/infiltration during or after storms.

# CONVERSION FACTORS
## U.S. Customary to SI (Metric)

| U.S. customary unit | | | SI | |
|---|---|---|---|---|
| Name | Abbreviation | Multiplier | Symbol | Name |
| acre | acre | 0.405 | ha | hectare |
| acre-foot | acre-ft | 1,233.5 | $m^3$ | cubic metre |
| acre-inch | acre-in. | 102.79 | $m^3$ | cubic metre |
| cubic foot | $ft^3$ | 28.32 | L | litre |
| | | 0.0283 | $m^3$ | cubic metre |
| cubic feet per minute | $ft^3/min$ | 0.0283 | $m^3/min$ | cubic metres per minute |
| cubic feet per minute per 100 gallons | $ft^3/min \cdot 100$ gal | 0.00747 | $m^3/min \cdot 100$ L | cubic metres per minute per 100 litres |
| cubic feet per pound | $ft^3/lb$ | 62.4 | L/kg | litres per kilogram |
| cubic feet per second | $ft^3/s$ | 28.32 | L/s | litres per second |
| cubic feet per square foot per minute | $ft^3/ft^2 \cdot min$ | 0.305 | $m^3/m^2 \cdot min$ | cubic metres per square metre per minute |
| cubic inch | $in.^3$ | 16.39 | $cm^3$ | cubic centimetre |
| | | 0.0164 | L | litre |
| cubic yard | $yd^3$ | 0.765 | $m^3$ | cubic metre |
| | | 764.6 | L | litre |
| degrees Fahrenheit | °F | 0.555 (°F-32) | °C | degrees Celsius |
| feet per minute | ft/min | 0.00508 | m/s | metres per second |
| feet per second | ft/s | 0.305 | m/s | metres per second |
| foot (feet) | ft | 0.305 | m | metre(s) |
| gallon(s) | gal | 3.785 | L | litre(s) |
| | | $3.785 \times 10^{-3}$ | $m^3$ | cubic metre |
| gallons per acre per day | gal/acre·d | 9.353 | L/ha·d | litres per hectare per day |
| gallons per capita per day | gal/capita·d | 3.785 | L/capita·d | litres per capita per day |
| gallons per day | gal/d | $4.381 \times 10^{-5}$ | L/s | litres per second |
| gallons per foot per minute | gal/ft·min | 0.207 | L/m·s | litres per metre per second |
| gallons per minute | gal/min | 0.0631 | L/s | litres per second |
| gallons per square foot | $gal/ft^2$ | 40.743 | $L/m^2$ | litres per square metre |
| gallons per square foot per day | $gal/ft^2 \cdot d$ | $1.698 \times 10^{-3}$ | $m^3/m^2 \cdot h$ | cubic metres per square metre per hour |
| | | 0.283 | $m^3/ha \cdot min$ | cubic metres per hectare per minute |
| gallons per square foot per minute | $gal/ft^2 \cdot min$ | 2.445 | $m^3/m^2 \cdot h$ | cubic metres per square metre per hour |
| | | 0.679 | $L/m^2 \cdot s$ | litres per square metre per second |
| horsepower | hp | 0.746 | kW | kilowatts |
| inch(es) | in. | 2.54 | cm | centimetre |
| inches per hour | in./h | 2.54 | cm/h | centimetres per hour |
| mile | mi | 1.609 | km | kilometre |
| million gallons | Mgal | 3.785 | ML | megalitres (litres $\times 10^6$) |
| | | 3785.0 | $m^3$ | cubic metres |
| million gallons per acre | Mgal/acre | 8353 | $m^3/ha$ | cubic metres per hectare |
| million gallons per acre per day | Mgal/acre·d | 0.039 | $m^3/m^2 \cdot h$ | cubic metres per square metre per hour |
| million gallons per day | Mgal/d | 43.808 | L/s | litres per second |
| | | 0.0438 | $m^3/s$ | cubic metres per second |
| million gallons per square mile | $Mgal/mi^2$ | 1.461 | $ML/km^2$ | megalitres per square kilometre |
| | | 1461 | $m^3/km^2$ | cubic metres per square kilometre |
| parts per billion | ppb | 1.0 | mg/L | micrograms per litre |
| parts per million | ppm | 1.0 | mg/L | milligrams per litre |
| pound(s) | lb | 0.454 | kg | kilogram(s) |
| | | 453.6 | g | gram(s) |
| pounds per acre per day | lb/acre·d | 1.121 | kg/ha·d | kilograms per hectare per day |
| pounds per cubic foot | $lb/ft^3$ | 16.018 | $kg/m^3$ | kilograms per cubic metre |
| pounds per 1000 cubic feet | $lb/1000\ ft^3$ | 16.018 | $g/m^3$ | grams per cubic metre |
| | | 0.016 | $kg/m^3$ | kilograms per cubic metre |
| pounds per mile | lb/mi | 0.282 | kg/km | kilograms per kilometre |
| pounds per million gallons | lb/Mgal | 0.120 | mg/L | milligrams per litre |
| pounds per square foot | $lb/ft^2$ | $4.882 \times 10^{-4}$ | $kg/cm^2$ | kilograms per square centimetre |
| | | 4.882 | $kg/m^2$ | kilograms per square metre |
| pounds per 1000 square feet per day | $lb/1000\ ft^2 \cdot d$ | $4.882 \times 10^{-3}$ | $kg/m^2 \cdot d$ | kilograms per square metre per day |
| pounds per square inch | $lb/in.^2$ | 0.0703 | $kg/cm^2$ | kilograms per square centimetre |
| square foot | $ft^2$ | 0.0929 | $m^2$ | square metre |
| square inch | $in.^2$ | 6.452 | $cm^2$ | square centimetre |
| square mile | $mi^2$ | 2.590 | $km^2$ | square kilometre |
| | | 259.0 | ha | hectare |
| square yard | $yd^2$ | 0.836 | $m^2$ | square metre |
| standard cubic feet per minute | std $ft^3/min$ | 1.699 | $m^3/h$ | cubic metres per hour |
| ton (short) | ton (short) | 0.907 | Mg (or t) | megagram (metric tonne) |
| tons per acre | tons/acre | 2240 | kg/ha | kilograms per hectare |
| tons per square mile | $tons/mi^2$ | 3.503 | kg/ha | kilograms per hectare |
| yard | yd | 0.914 | m | metre |